ANALYSIS AND CAPACITY BASED EARTHQUAKE RESISTANT DESIGN OF MULTI BAY MULTI STOREYED 3D-RC FRAME.

A THESIS SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

Master in Technology

in

Civil Engineering

By

RASMI RANJAN SAHOO

Department of Civil Engineering

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Under the Guidance of

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CERTIFICATE

This is to certify that the work in this thesis entitled "**ANALYSIS AND CAPACITY BASED EARTHQUAKE RESISTANT DESIGN OF MULTI BAY MULTI STOREYED 3D-RC FRAME.**" submitted by Shri. **Rasmi Ranjan Sahoo** in partial fulfillment of the requirements for the award of Master of Technology Degree in **Civil Engineering** with specialization in "**Structural Engineering**" at the National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance during session 2007-2008

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other university or institute for the award of any Degree or Diploma.

Date: 30^{th} May , 2008

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CONTENTS

ABSTRACT

Earthquakes in different parts of the world demonstrated the disastrous consequences and vulnerability of inadequate structures. Many reinforced concrete (RC) framed structures located in zones of high seismicity in India are constructed without considering the seismic codal provisions. The vulnerability of inadequately designed structures represents seismic risk to occupants.

The main cause of failure of multi-storey multi-bay reinforced concrete frames during seismic motion is the soft storey sway mechanism or column sway mechanism. The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make beams to be the ductile weak links than columns. This method of designing RC buildings is called the strong-column weak-beam design method.

If the frame is designed on the basis of strong column-weak beam concept the possibilities of collapse due to sway mechanisms can be completely eliminated. In multi storey frame this can be achieved by allowing the plastic hinges to form, in a predetermined sequence only at the ends of all the beams while the columns remain essentially in elastic stage and by avoiding shear mode of failures in columns and beams. This procedure for design is known as Capacity based design which would be the future design philosophy for earthquake resistant design of multi storey multi bay reinforced concrete frames.

The aim of this project work is to present a detailed worked out example on 3 dimensional seismic analysis and capacity based design of five storied-three bay reinforced concrete frame.

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Chapter - 1

Introduction

INTRODUCTION:

Civil engineering structures are mainly designed to resist static loads. Generally the effects of dynamic loads acting on the structure are not considered. This feature of neglecting the dynamic forces sometimes becomes the cause of disaster, particularly in case of earthquake. The resent example of this category is Bhuj earthquake occurred on Jan.26, 2001. This has created a growing interest and need for earthquake resistant design of structures.

Conventional Civil Engineering structures are designed on the basis of strength and stiffness criteria. The strength is related to ultimate limit state, which assures that the forces developed in the structure remain in elastic range. The stiffness is related to serviceability limit state which assures that the structural displacements remains within the permissible limits. In case of earthquake forces the demand is for ductility. Ductility is an essential attribute of a structure that must respond to strong ground motions. Ductility is the ability of the structure to undergo distortion or deformation without damage or failure which results in dissipation of energy. Larger is the capacity of the structure to deform plastically without collapse, more is the resulting ductility and the energy dissipation. This causes reduction in effective earthquake forces.

The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The correct building components need to be made ductile. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make beams to be the ductile weak links than columns. This method of designing RC buildings is called the strong-column weak-beam design method.

Most of the energy developed during earthquake is dissipated by columns of the soft stories. In this process the plastic hinges are formed at the ends of columns, which transform the soft storey into a mechanism. In such case the collapse is unavoidable. Therefore, the soft stories deserve a special consideration in analysis and design.

CHAPTER -2

Literature Review

Literature Review:

EERI (2002), survey the Bhuj earthquake or Kutch earthquake. The earthquake ranks as one of the most destructive events recorded so far in India in terms of death toll, damage to infrastructure and devastation in the last fifty years. The major cities affected by the earthquake are Bhuj, Anjar, Bhachau, Gandhidham, Morbi, Rajnagar etc. where majority o the causalities and damages occurred. Various types of structures reveal weakness in the form of design and planning practices, inadequate analysis, design deficiency and even poor quality of construction.

Reinforced concrete multi-storied buildings in India for the first time have been subjected to a strong ground motion shaking in Bhuj earthquake (January 26, 2001). It has been observed that the principal reasons of failure may be accounted to soft stories, floating columns, mass irregularities, poor quality of construction material and faulty construction practices etc.

The building framing system is generally moment resisting, consisting of reinforced concrete slabs cast monolithically with beams and columns on sallow isolated footing. The upper floors are generally constructed with infill walls made of unreinforced bricks, cut stones or cement concrete blocks. In major commercial cities, the ground floor/basement is often used for commercial and parking purposes, where the infill walls are omitted, resulting in soft or weak stories. Most of the buildings have overhanging covered balconies of about 1.5 m span on higher floors. The architects often erect a heavy beam from the exterior columns of the building to the end of the building on the first floor onwards. A principal beam is provided at the end of the erected girder to create more parking spaces at the ground floor and allowing more space on the upper floors. The upper floor balconies or other constructions are constructed on the peripheral beams. The infill walls, which are present in the upper floors and absent in the ground floor, create a floating box type situation.

Columns in the most of the buildings are of uniform size along the height of the buildings, with marginal change in the grade of concrete and reinforcement in the ground floor. It is apparent that the columns are designed only for axial load, without considering the effect of framing action and lateral loads. The ground floor columns are not cast up to the bottom of the beam and gap of 200 mm 250 mm is left called as “topi” to accommodate the beam reinforcement, which makes the construction more vulnerable. Due to congestion of reinforcement in this region, the compaction of concrete is not properly done which results in poor quality of concrete and honeycombing. The longitudinal reinforcement is often lap-spliced just above the floor slab. The spacing of transverse reinforcement over lap splice is same as elsewhere in the column rather being closely spaced. There is no sign of special confinement reinforcement and ductile detailing in the columns. This is a faulty design practice from seismic point if view.

The foundation in private buildings generally consists of an isolated footing with a depth of about 1.5 m for G+3 buildings and 2.7 m to 3.5 m for G+10 buildings. The plan sizes of footings are usually 1.2 m × 1.2 m, 1.8 m × 1.8 m or 2.4 m × 2.4 m. There are no tie beams interconnecting the footing, and plinth beams connecting the column at the ground storey level.

Identification of damage in RC buildings in Bhuj earthquake:

Goel (2001), observed reinforced concrete buildings have been damaged on a large scale in Bhuj earthquake of January 26, 2001. The buildings have been damaged due to various reasons. Identification of a single cause of damage to buildings is not possible. There are combine reasons, which are responsible for multiple damages.

Soft storey failure

In general, multi-storied buildings in metropolitan cities require open taller first storey for parking of vehicles or for retail shopping, large space for meeting room or a banking hall owing to lack of horizontal space and high cost. Due to this functional requirement, the first storey has lesser strength and stiffness as compared to upper stories, which are stiffened by masonry infill walls. This characteristic of building construction creates “weak” or “soft” storey problems in multi-storey buildings. Increased flexibility of first storey results in extreme deflections, which in turn, leads to concentration of forces at second storey connections accompanied by large plastic deformations. In addition, most of the energy developed during earthquake is dissipated by columns of the soft stories. In this process the plastic hinges are formed at the ends of columns, which transform the soft storey into a mechanism. In such case the collapse is unavoidable. Therefore, the soft stories deserve a special consideration in analysis and design.

It has been observed from the survey that the damage is due to collapse and buckling of columns especially where parking places are not covered properly. On the contrary, the damage is reduced considerably where the parking spaces are covered adequately. It is recognized that this type of failure results from the combination of several other unfavorable reasons, such as torsion, excessive mass on upper floors, p-Δ effects and lack of ductility in the bottom storey.

Typical soft storey failure in Bhuz

Soft storey failure in Morbi

Figure 1

Floating columns

Most of buildings in Ahmedabad and Gandhidham, are covering the maximum possible area on a plot within available bylaws. Since balconies are not counted in Floor Space Index (FSI), building have balconies overhanging in the upper stories beyond the column foot print area at the ground storey, overhangs upto 1.2 m to 1.5 m in plan are usually provided on each side of the building. In the upper stories, the perimeter columns of ground storey are discontinued, and floating columns are provided along the overhanging perimeter of the building. The floating column rest at the tip of the taper overhanging beams without considering the increased vulnerability of lateral load resisting system due to vertical discontinuity. This type of construction does not create any problem under vertical loading conditions. But during an earthquake a clear load path is not available for transforming lateral forces to foundation. Lateral forces accumulated in upper floors during the earthquake have to be transmitted by the projected cantilever beams. Overturning forces thus developed overwhelm the columns of the ground floor. Under this situation the columns begin to deform and buckle, resulting in total collapse. This is because of primary deficiency in the strength of ground floor columns, projected cantilever beam and ductile detailing of beam-column joints. Ductile connection at the exterior beam-column joint is indispensable for transferring these forces.

Failure of floating column

Figure2

Damage to structural element:

Oblong cross section, a space left at the top of column called “topi” during casting and relatively slender column sections compared with beam sections are the main structural defects in columns. These columns are neither designed nor detailed for ductility. Lack of confinement due to large tie spacing, insufficient development length, inadequate splicing of all column bars at the same section, hook configurations of reinforcement do not comply with ductile detailing practices. Localized failure at the top and bottom of column is due to inadequate spacing of ties in critical areas and the presence of strong beams.

Effect of earthquake on code designed structures:

The Bureau of Indian standards (BIS) has published two codes IS 1893 (Part 1): 2002 and IS 13920: 1993 for earthquake resistant design of reinforced concrete buildings. The former code deals with the determination of forces and general considerations for design of buildings while latter code deals with the detailing of reinforced concrete structures for ductility. The government buildings follow the design code as a mandatory requirement. Therefore, the performance of governmental buildings in the Bhuj earthquake has ben better on account of code compliance. The multi-storied (G+9) reinforced concrete building, residential quarters for regional passport office and Ayakar Bhawn (G+3) RC building with part basement at Ahmedabad were constructed by central public works department (CPWD) in the years 2000 and 1954 respectively. These two buildings sustained minor damage in the form of cracking of infill brick wall. Both buildings were in working condition after the earthquake and were not required to be vacated.

Thus the design of buildings should be based on seismic codes. The multi-storied reinforced buildings with vertical irregularities like soft storey construction and the buildings with floating column should be designed on the basis of earthquake analysis.

Methods of Seismic Design:

Based on the three criteria strength, stiffness and ductility the methods for seismic design are described below:

LATERAL STRENGTH BASED DESIGN:

This is most common seismic design approach adopted nowadays. It is based on providing the structure with the minimum lateral strength to resist seismic loads, assuming that the structure will behave adequately in the non-linear range. For this reason only some simple construction detail rules are needed to be satisfied.

DISPLACEMENT BASED DESIGN:

In this method the structure is designed to possess adequate ductility so that it can dissipate energy by yielding and survive the shock. This method operates directly with deformation quantities hence gives better insight on the expected performance of the structures. The displacement based design approach has been adopted by the seismic codes of many countries.

CAPACITY BASED DESIGN:

In this design approach the structures are designed in such a way so that plastic hinges can form only in predetermined positions and in predetermined sequences. The concept of this method is to avoid brittle mode of failure. This is achieved by designing the brittle modes of failure to have higher strength than ductile modes.

ENERGY BASED DESIGN:

This is the most promising and futuristic approach of earthquake resistant design. In this approach it is assume that the total energy input is collectively resisted by kinetic energy, the elastic strain energy and energy dissipated through plastic deformations and damping.

Seismic Analysis Procedures:

Main features of seismic method of analysis based on Indian Standard 1893(part 1): 2002 are described as follows

Equivalent lateral force method:

The Equivalent lateral force method is the simplest method of analysis and requires less computational effort because the forces depend on the code based fundamental period of structures with some empirical modifier. The design base shear shall first be computed as a whole, and then be distributed along the height of buildings based on simple formulae appropriate for buildings with regular distribution of mass and stiffness. The design lateral force obtained at each floor level shall be distributed to individual lateral load resisting elements depending upon floor diaphragm action.

The design lateral force or design base shear and the distribution are given by some empirical formulae given in the I.S 1893.

Response Spectrum analysis:

This method is applicable for those structures where modes other than the fundamental one affect significantly the response of the structure. In this method the response of Multi degree of freedom system is expressed as the superposition of modal response, each modal response being determined from the spectral analysis of Single–degree of freedom system, which is then combined to compute the total response.

Elastic Time history analysis:

A linear analysis, time history analysis over comes all disadvantages of modal response spectrum provided non linear behavior is not involved. The method requires greater computational efforts for calculating the response at discrete times. One interesting advantage of this is that the relative signs of response quantities are preserved in the response histories.

Chapter -3

Capacity Based Design

CAPACITY BASED DESIGN:

Capacity Design is a concept or a method of designing flexural capacities of critical member sections of a building structure based on a hypothetical behavior of the structure in responding to seismic actions. This hypothetical behavior is reflected by the assumptions that the seismic action is of a static equivalent nature increasing gradually until the structure reaches its state of near collapse and that plastic hinging occurs simultaneously at predetermined locations to form a collapse mechanism simulating ductile behavior. The actual behavior of a building structure during a strong earthquake is far from that described above, with seismic actions having a vibratory character and plastic hinging occurring rather randomly. However, by applying the Capacity Design concept in the design of the flexural members of the structure, it is believed that the structure will possess adequate seismic resistance, as has been proven in many strong earthquakes in the past.

A feature in the Capacity Design concept is the ductility level of the structure, expressed by the displacement ductility factor or briefly ductility factor. This is the ratio of the lateral displacement of the structure due to the Design Earthquake at near collapse and that at the point of first yielding.

The basic of capacity based design lies on strong column and weak beam concept. The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The correct building components need to be made ductile. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make beams to be the ductile weak links than columns. This method of designing RC buildings is called the strong-column weak-beam design method.

Basic steps for capacity based design:

1. Design loads i.e. dead loads, live loads and earthquake loads are calculated.
2. Seismic analysis of the frame for all load combination specified in IS 1893 (Part I):2002 are done.
3. Members are designed (as per IS 456:200) for maximum forces obtained from all load combinations. Beams are designed for maximum sagging and maximum hogging moments. Provided reinforcements are calculated following the norms given in code.

Columns are designed for the combination for moment and corresponding axial force providing maximum interaction effect i.e. considering the eccentricity.

4. The flexural capacities of the beams under sagging and hogging condition for the provided reinforcements are calculated.
5. The flexural capacity of columns at a joint is compared with actual flexural capacity of joining beams. If the sum of capacities of columns is less than the sum of capacities of beams multiplied by over strength factor, the column moments should be magnified by the factor (moment magnification factor) by which they are lacking in moment capacity over beams. If the sum of the column moments is greater than sum of beam moments, there is no need to magnify the column moments.
6. Columns are designed for the revised moments and the axial force coming on it from the analysis.
7. Shear capacity of beams are calculated on the basis or their actual moment capacities and shear reinforcements are calculated.
8. Similarly shear capacity of column is calculated on the basis of magnified moment capacities. Then the columns are designed for shear.

STEP-BY-STEP PROCEDURE FOR CAPACITY BASED DESIGN:

Step 1: Seismic Analysis of Frame (G+3)

Seismic analysis of the plane frame is carried out with all load combinations as per IS 1893 (Part 1); 2002. The maximum interaction effect for columns and maximum force for beams from all load combinations for each member is considered for design. Design forces in columns and beams are presented in Figures 7 & 8. In capacity based design, beams are designed similar to normal design procedure for the calculated forces by the linear elastic analysis for different load combinations. Figure below shows the actual amount of longitudinal reinforcement in the beams.

The design forces of columns are not completely based on linear elastic analysis rather they depend upon the actual flexural capacities of the beams framing into the same joint. So that plastic hinges may not form at the base of column above and at the top of the column below the joint.

Step 2: Determination of Flexural Capacity of Beam

The flexural capacities of the beams under hogging and sagging conditions for the provided reinforcement are calculated.

Step 3: Establishing a strong Column-weak Beam mechanism

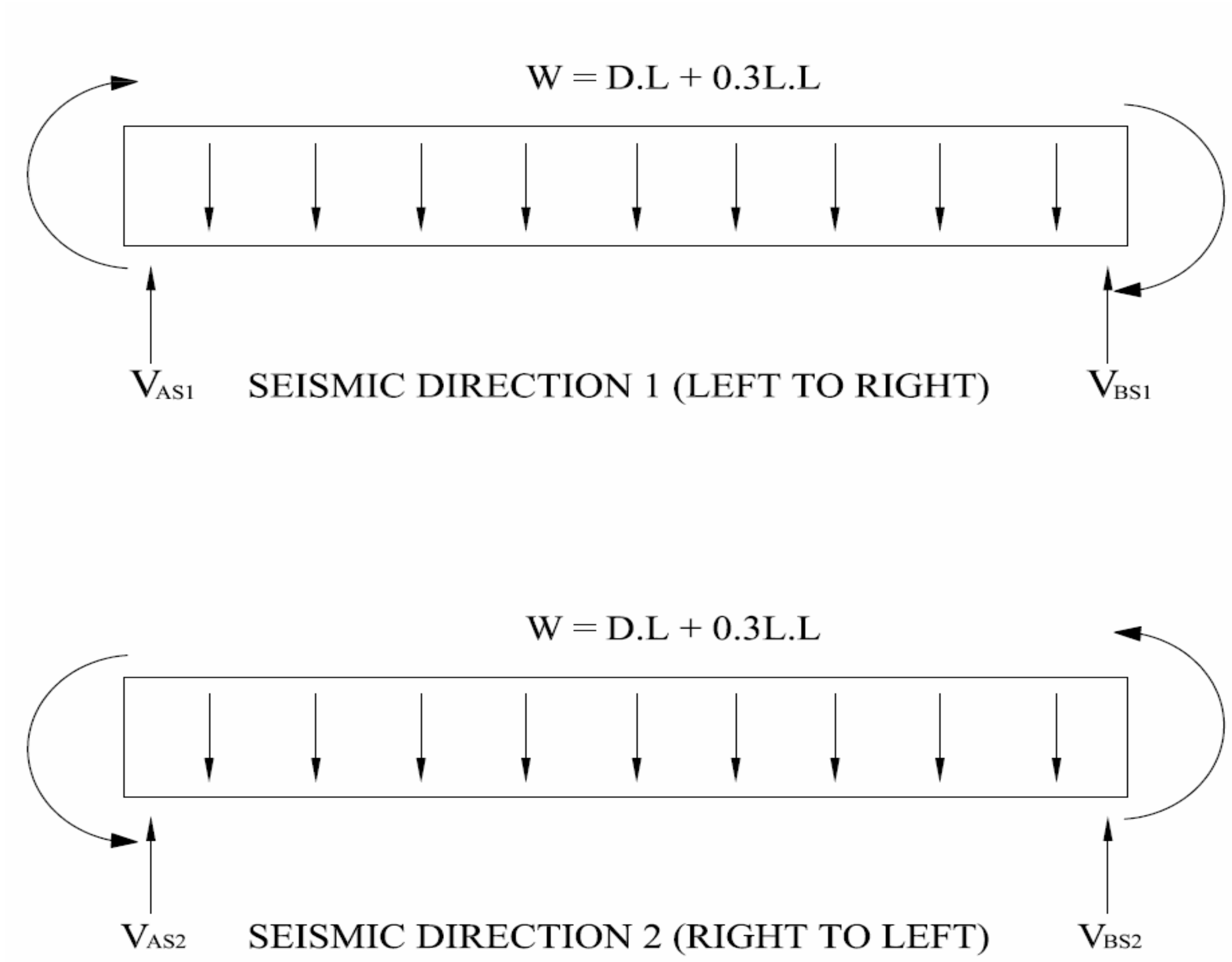
To eliminate the possibility of a column sway mechanism (soft storey) during the earthquake, it is essential that the plastic hinges should be formed in beams (except at the base of the columns of ground storey). This condition can be achieved after moment capacity verification of columns with beams at every joint of the frame with the formation of beam mechanism only. The amount by which the design moments of columns at a joint to be magnified, is achieved by determination of the moment magnification factor at that particular joint.

Step 4: Determination of Moment Magnification Factors for Columns

The moment capacities of columns are to be checked for the sum of the moment capacities of beams at the joint with an over strength factor of 1.4. If the ″sum of capacities of columns″ is less than the "sum of moment capacities of beams multiplied by over strength factor", the column moments should be magnified by the factor by which they are lacking in moment capacity over beams. If the sum of column moments is greater than sum of beam moments, there is no need to magnify the column moments. In such cases the multiplying factor is taken as unity. After obtaining the foment magnification factors, the column flexural strengths are to be increased accordingly at every joint and the maximum revised moment from the top and bottom joints to be taken for design.

Step 5: Capacity Design for Shear in Beams

The design shear forces in beams are corresponding to the equilibrium condition of the beam under the appropriate gravity load (permanent dead load + % of live load) and to end resisting moments corresponding to the actual reinforcement provided, further multiplied by a factor γ_{Rd} (Figure below). This γ_{Rd} factor compensates the partial safety factor y_s applied to yield strength of steel and to account the strain hardening effects. In the absence of more reliable data, γ_{Rd} may be taken as 1.4



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Equilibrium condition for the determination of shear force

Figure-3

The shear force is in both the directions is determined by the following equation.

$V_{AS1} = wL/2 - \gamma_{Rd} (M_{AR}+M_{BR})/L$

$V_{BS1}= wL/2+\gamma_{Rd} (M_{AR}+M_{BR})/L$

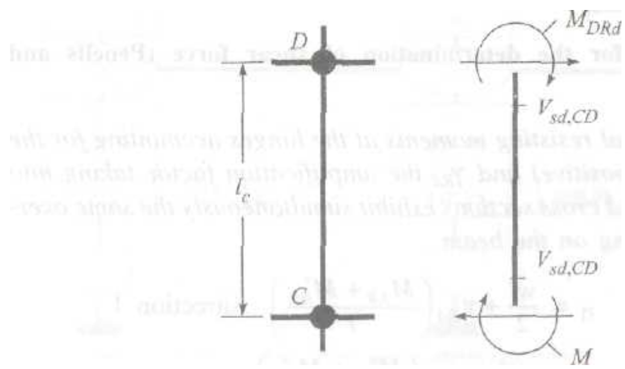
$V_{AS2} = wL/2 + \gamma_{Rd} (M^1_{AR}+M^1_{BR})/L$

$V_{BS2} = wL/2 - \gamma_{Rd} (M^1_{AR}+M^1_{BR})/L$

Where M_{AR}, M_{BR}, M^1_{AR}, M^1_{BR} are the actual resisting moments of the beam in seismic direction 1 and 2 respectively. γ_{Rd} is the amplification factor and w comprises of the dead and live load.

Step 6: Capacity Design for Shear in Columns

Capacity design shear forces are evaluated by considering the equilibrium of the column under the actual resisting moments at its ends.



Capacity design values of shear forces acting on columns

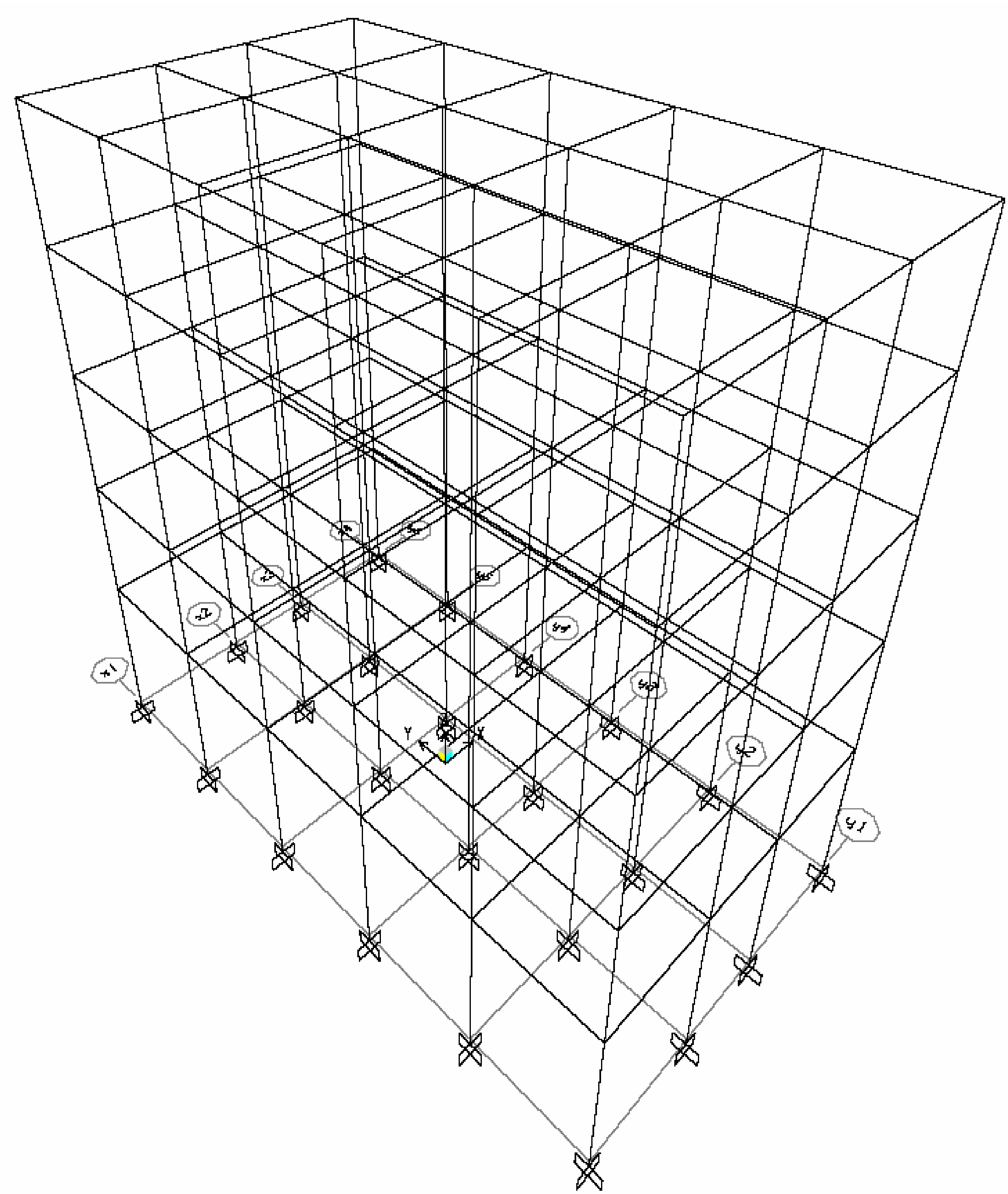
Figure-4

Here M_{DRd} and M_{CRd} are the flexural capacities of the end sections and l_c is the clear height of the column.

CHAPTER -4

Analysis of 3D-RC Frame using SAP 2000

Analysis of 3D-RC frame using SAP:

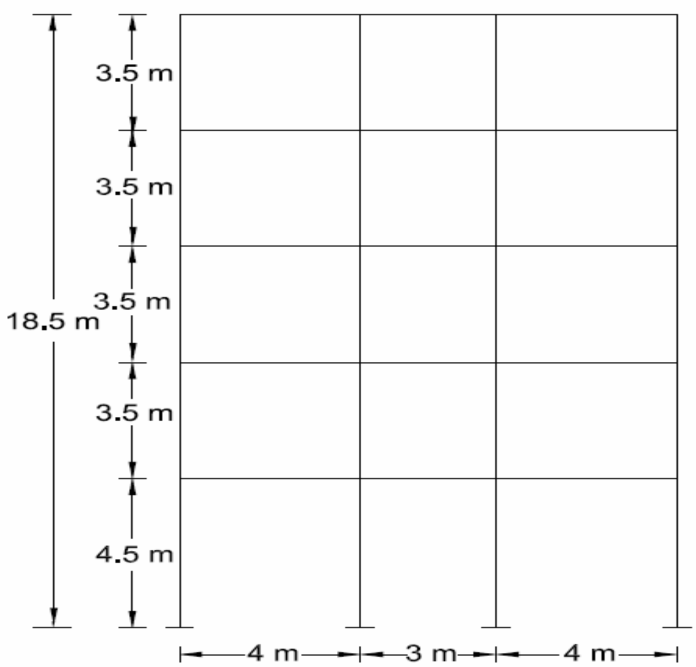


Multi bay multi storied RC Frame
Figure-5

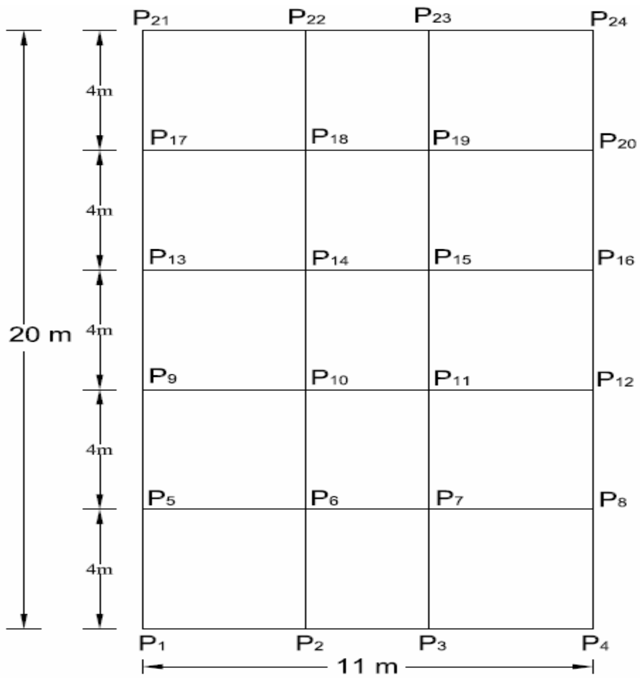
Problem Statement:

A G+4 building is taken for analysis. The salient features of the building are:

1.	Type of structure	--- multi storey rigid joint frame.
2.	seismic zone	--- IV
3.	Type of soil	--- Medium
4.	No. of stories	--- (G+4)
5.	Imposed Load	---3.5 KN/m^2
6.	Terrace water proofing	--- 1.5 KN/m^2
7.	Floor finishes	--- 0.5 KN/m^2
8.	Depth of slab	--- 120 mm
9.	Materials	--- M 20 concrete and Fe 415 steel
10.	Unit weight of RCC	--- 25 KN/m^2
11.	Beams	--- 300 × 450 mm
12.	Columns	--- 300 × 500 mm (outer) in XZ plane.
		--- 300 × 550 mm (internal) in XZ plane.
16.	Clear cover of beam	--- 25 mm
17.	Clear cover of column	---40 mm
18.	Wall thickness	--- 250mm.



Elevation in XZ Plane
Figure-6



Plan

Figure-7

<u>Calculation of seismic coefficient, A_h:</u>

As per IS1893 (Part 1): 2002

$A_h = \frac{ZI}{2R}\left(\frac{S_a}{g}\right)$ (Clause 6.4.2 IS1893 (Part 1): 2002)

Where

Z = Zone factor.

For zone IV

Z = 0.24

I = Importance factor.

= 1.5 for public building.

R = Response reduction factor.

= 5 for special RC moment resisting frame.

$\left(\frac{S_a}{g}\right)$ = Average response acceleration coefficient and depends upon fundamental natural period.

$T_a = \frac{.09h}{\sqrt{d}}$ (Clause 7.6.2 IS1893 (Part 1): 2002)

Where

T_a = Fundamental natural period.

h = Height of the building in meter.

d = Base dimension of the building at plinth level in m along the considered direction of force.

Here

h = 18.5 m.

d = 11 m. along X direction.

d = 20 m. along Y direction.

$$T_a = \frac{.09 \times 18.5}{\sqrt{11}}$$

= .502 sec. along X direction.

$$T_a = \frac{.09 \times 18.5}{\sqrt{20}}$$

= 0.372 sec. along Y direction.

As the soil type is medium

$\left(\frac{S_a}{g}\right)$ = 2.5 in both directions. (Clause 6.4.5 IS1893 (Part 1): 2002)

$$A_h = \frac{0.24 \times 1.5}{2 \times 5} \times (2.5)$$

= .09.

<u>Determination of loads:</u>

Dead load calculations:

The dead loads on various beams and columns in the frame are calculated as follows

<u>**Dead load at roof level**</u>	<u>**Dead load at floor level**</u>
Weight of the slab: Total intensity of slab including floor finish and terrace waterproofing = (0.12 × 25 + 1.5 + 0.5) = 5 KN / m. **For Beam No: - P_1P_2, $P_{21}P_{22}$, P_3P_4, $P_{23}P_{24}$,**	**Weight of the slab:** Total intensity of slab including floor finish = (0.12 × 25 + 0.5) = 3.5 KN/m. **For Beam No: - P_1P_2, $P_{21}P_{22}$, P_3P_4, $P_{23}P_{24}$,**

P_1P_5, P_5P_9, P_9P_{13}, $P_{17}P_{21}$, P_4P_8, P_8P_{12}, $P_{12}P_{16}$, $P_{16}P_{20}$, $P_{20}P_{24}$. Tributary floor area = $\frac{1}{2}(4\times 2)=4\,m^2$ Slab load on beam = 4 × 5 = 20 KN. Load on beam per meter = $\frac{20}{4}=5$ KN/m. Parapet wall load on beam = 20 × 0.1 × 1 = 2 KN/m Total load on beam : = 5 + 2 = 7 KN/m.	**P_1P_5, P_5P_9, P_9P_{13}, $P_{17}P_{21}$, P_4P_8, P_8P_{12}, $P_{12}P_{16}$, $P_{16}P_{20}$, $P_{20}P_{24}$.** Tributary floor area = $\frac{1}{2}(4\times 2)=4\,m^2$ Slab load on beam = 4 × 3.5 = 14 KN. Load on beam per meter = $\frac{14}{4}=3.5$ KN/m. Wall load on beam = 20 × 0.25 (3.5 – 0.45) = 15.25 KN/m. Total load on beam : = 3.5 + 15.25 = 18.75 KN/m.
For Beam No: - P_5P_6, P_9P_{10}, $P_{13}P_{14}$, $P_{17}P_{18}$, P_7P_8, P_9P_{13}, $P_{11}P_{12}$, $P_{15}P_{16}$, $P_{19}P_{20}$. Tributary floor area = $\left[\frac{1}{2}(4\times 2)\right]\times 2=8\,m^2$ Slab load on beam = 8 × 5 = 40 KN. Load on beam per meter = $\frac{40}{4}=10$ KN/m. Parapet wall load on beam = 20 × 0.1 × 1 = 2 KN/m Total load on beam : = 10 + 2 = 12 KN/m.	**For Beam No: - P_5P_6, P_9P_{10}, $P_{13}P_{14}$, $P_{17}P_{18}$, P_7P_8, P_9P_{13}, $P_{11}P_{12}$, $P_{15}P_{16}$, $P_{19}P_{20}$.** Tributary floor area = $\left[\frac{1}{2}(4\times 2)\right]\times 2=8\,m^2$ Slab load on beam = 8 × 3.5 = 28 KN. Load on beam per meter = $\frac{28}{4}=7$ KN/m. Wall load on beam = 20 × 0.25 (3.5 – 0.45) = 15.25 KN/m. Total load on beam : = 7 + 15.25 = 22.25 KN/m.
For Beam No: - P_2P_6, P_6P_{10}, $P_{10}P_{14}$, $P_{14}P_{18}$, $P_{18}P_{22}$, P_3P_7, P_7P_{11}, $P_{11}P_{15}$, $P_{15}P_{19}$, $P_{19}P_{23}$. Tributary floor area = $\frac{1}{2}(4\times 2)+\frac{1}{2}(1+4)\times 1.5=7.75\,m^2$	**For Beam No: - P_2P_6, P_6P_{10}, $P_{10}P_{14}$, $P_{14}P_{18}$, $P_{18}P_{22}$, P_3P_7, P_7P_{11}, $P_{11}P_{15}$, $P_{15}P_{19}$, $P_{19}P_{23}$.** Tributary floor area = $\frac{1}{2}(4\times 2)+\frac{1}{2}(1+4)\times 1.5=7.75\,m^2$

Slab load on beam = 7.75 × 5 = 38.75 KN. Load on beam per meter = $\frac{38.75}{4}$ = 9.69 KN/m. Parapet wall load on beam = 20 × 0.1 × 1 = 2 KN/m Total load on beam : = 9.69 + 2 = 11.69 KN/m.	Slab load on beam = 7.75 × 3.5 = 27.125 KN. Load on beam per meter = $\frac{27.125}{4}$ = 6.78 KN/m. Wall load on beam = 20 × 0.25 (3.5 – 0.45) = 15.25 KN/m. Total load on beam : = 6.78 + 15.25 = 22.03 KN/m.
For Beam No: - P_2P_3, $P_{22}P_{23}$. Tributary floor area = $\frac{1}{2}(4\times1.5) = 2.25\,m^2$ Slab load on beam = 2.25 × 5 = 11.25 KN. Load on beam per meter = $\frac{11.25}{3}$ = 3.75 KN/m. Parapet wall load on beam = 20 × 0.1 × 1 = 2 KN/m Total load on beam : = 3.75 + 2 = 5.75 KN/m.	**For Beam No: - P_2P_3, $P_{22}P_{23}$.** Tributary floor area = $\frac{1}{2}(4\times1.5) = 2.25\,m^2$ Slab load on beam = 2.25 ×3.5 = 7.875 KN Load on beam per meter = $\frac{7.875}{3}$ = 2.625 KN/m. Wall load on beam = 20 × 0.25 (3.5 – 0.45) = 15.25 KN/m. Total load on beam : = 2.625 + 15.25 = 17.875 KN/m.
For Beam No: - P_6P_7, $P_{10}P_{11}$, $P_{14}P_{15}$, $P_{18}P_{19}$. Tributary floor area = $\left[\frac{1}{2}(3\times1.5)\right]\times 2 = 4.5\,m^2$ Slab load on beam = 4.5 × 5 = 22.5 KN. Load on beam per meter = $\frac{22.5}{3}$ = 7.5 KN/m.	**For Beam No: - P_6P_7, $P_{10}P_{11}$, $P_{14}P_{15}$, $P_{18}P_{19}$.** Tributary floor area = $\left[\frac{1}{2}(3\times1.5)\right]\times 2 = 4.5\,m^2$ Slab load on beam = 4.5 × 3.5 = 15.75 KN. Load on beam per meter = $\frac{15.75}{3}$ = 5.25 KN/m.

Parapet wall load on beam = 20 × 0.1 × 1 = 2 KN/m Total load on beam : = 7.5 + 2 = 9.5 KN/m.	Wall load on beam = 20 × 0.25 (3.5 – 0.45) = 15.25 KN/m. Total load on beam : = 5.25 + 15.25 = 20.5 KN/m.

Imposed load calculations:

The imposed loads on various beams and columns in the frame are calculated as follows

For Beam No: - P_1P_2, $P_{21}P_{22}$, P_3P_4, $P_{23}P_{24}$, P_1P_5, P_5P_9, P_9P_{13}, $P_{17}P_{21}$, P_4P_8, P_8P_{12}, $P_{12}P_{16}$, $P_{16}P_{20}$, $P_{20}P_{24}$.

Tributary floor area = $\frac{1}{2}(4 \times 2) = 4\,\text{m}^2$

Total load on beam: 3.5 × 4 = 14 KN.

Load on beam per meter = $\frac{14}{4} = 3.5$ KN/m.

For Beam No: - P_5P_6, P_9P_{10}, $P_{13}P_{14}$, $P_{17}P_{18}$, P_7P_8, P_9P_{13}, $P_{11}P_{12}$, $P_{15}P_{16}$, $P_{19}P_{20}$.

Tributary floor area = $\left[\frac{1}{2}(4 \times 2)\right] \times 2 = 8\,\text{m}^2$

Total load on beam: 3.5 × 8 = 28 KN.

Load on beam per meter = $\frac{28}{4} = 7$ KN/m.

For Beam No: - P_2P_6, P_6P_{10}, $P_{10}P_{14}$, $P_{14}P_{18}$, $P_{18}P_{22}$, P_3P_7, P_7P_{11}, $P_{11}P_{15}$, $P_{15}P_{19}$, $P_{19}P_{23}$.

Tributary floor area = $\frac{1}{2}(4 \times 2) + \frac{1}{2}(1 + 4) \times 1.5 = 7.75\,\text{m}^2$

Total load on beam: 3.5 × 7.75 = 27.125 KN.

Load on beam per meter = $\frac{27.125}{4} = 6.78$ KN/m.

For Beam No: - P_2P_3, $P_{22}P_{23}$.

Tributary floor area = $\frac{1}{2}(4 \times 1.5) = 2.25\,m^2$

Total load on beam = 2.25 × 3.5 = 7.875 KN.

Load on beam per meter = $\frac{7.875}{3} = 2.625$ KN/m.

For Beam No: - P_6P_7, $P_{10}P_{11}$, $P_{14}P_{15}$, $P_{18}P_{19}$.

Tributary floor area = $\left[\frac{1}{2}(3 \times 1.5)\right] \times 2 = 4.5\,m^2$

Slab load on beam = 4.5 × 3.5 = 15.75 KN.

Load on beam per meter = $\frac{15.75}{3} = 5.25$ KN/m.

Earthquake load calculations:

Determination of total base shear:

Dead load:

a) Weight of floor (W_s + FF)

= 20 × 11 × (3 + 0.5)

= 770 KN.

b) Weight of roof (W_s + TWs + FF)

= 20 × 11 × (3 + 1.5 + 0.5)

= 1100 KN.

c) Weight of peripheral beams (Transverse)

$$= \left[\left\{2 \times \left(4 - \frac{0.5}{2} - \frac{0.55}{2}\right)\right\} \times 2 + \left\{1 \times \left(3 - \frac{0.5}{2} - \frac{0.55}{2}\right)\right\} \times 2\right] \times 3.375$$

= 49.35 KN.

d) Weight of peripheral beams (Longitudinal)

$$= \left[\left\{ 5 \times \left(4 - \frac{0.5}{2} - \frac{0.55}{2} \right) \right\} \times 2 \right] \times 3.375$$

= 97.125 KN.

e) Weight of Parapet wall

$$= \left[2 \times (20 + 11) \times 1 \times 0.1 \times 20 \right]$$

= 124 KN.

f) Weight of external wall

= [20 × 0.25 × (37 + 18.9) × (3.5 – 0.45)]

= 852.475 KN.

g) Interior beams (Transverse)

$$= \left[\left\{ 2 \times \left(4 - \frac{0.5}{2} - \frac{0.55}{2} \right) \right\} \times 2 + \left\{ 1 \times \left(3 - \frac{0.5}{2} - \frac{0.55}{2} \right) \right\} \times 4 \right] \times 3.375$$

= 98.7 KN.

h) Interior beams (Longitudinal)

$$= \left[\left\{ 5 \times \left(4 - \frac{0.5}{2} - \frac{0.55}{2} \right) \right\} \times 2 \right] \times 3.375$$

= 97.125 KN.

i) Weight of Interior Walls

= 20 × 0.25 × (38.6 + 37) × 3.05

= 1152.9 KN

j) Weight of exterior column/Height

= 2 × 6 × 0.4 × 0.5 × 25

= 60 KN/m

k) Weight of interior column/Height

= 2 × 6 × 0.4 × 0.55 × 25

= 66 KN/m

Imposed load:

Imposed load on roof = Zero.

Imposed load on floor = 50% of Imposed load

= 0.5 × 3.5 KN/m^2

= 1.75 KN/m^2

Total Imposed load on each floor = 20 × 11 × 1.75

= 385 KN.

<u>Concentrated mass:</u>

i) At roof :

$$= \left[b + c + d + e + \frac{f}{2} + g + h + \frac{i}{2} + \left(\frac{j \times 3.5}{2} \right) + \left(\frac{k \times 3.5}{2} \right) + 0 \right]$$

$$= \left[\begin{array}{l} 1100 + 49.35 + 97.125 + 124 + \dfrac{882.475}{2} + 98.7 + 97.125 + \dfrac{1152.9}{2} \\ + \left(\dfrac{60 \times 3.5}{2} \right) + \left(\dfrac{49.5 \times 3.5}{2} \right) \end{array} \right] + 0$$

= 2734.3625 KN

ii) At 4th, 3rd, 2nd floor:

$$= \left[a + c + d + f + g + h + i + (j + k) \times 3.5 \right] + 385$$

$$= \left[\begin{array}{l} 770 + 49.35 + 97.125 + 852.475 + 98.7 + 97.125 + 1152.9 \\ + \dfrac{(60 + 49.5)}{2} \end{array} \right] + 385$$

= 3833.425 KN

iii) At 1st floor:

$$= \left[a + c + d + f + g + h + i + \frac{(j + k) \times (3.5 + 4)}{2} \right] + 385$$

$$= \left[\begin{array}{l} 770 + 49.35 + 97.125 + 852.475 + 98.7 + 97.125 + 1152.9 \\ + \dfrac{(60 + 49.5) \times (3.5 + 4)}{2} \end{array} \right] + 385$$

= 3857.05 KN

Total weight = 2734.3625 + 3 × 3833.425 + 3857.05

= 18091.69 KN.

Total base shear = A_h × W

= 0.09 × 18091.69

= 1628.2521 KN.

If earthquake force will act in X direction, then base shear in each frame

$$= \frac{1628.2521}{6}$$

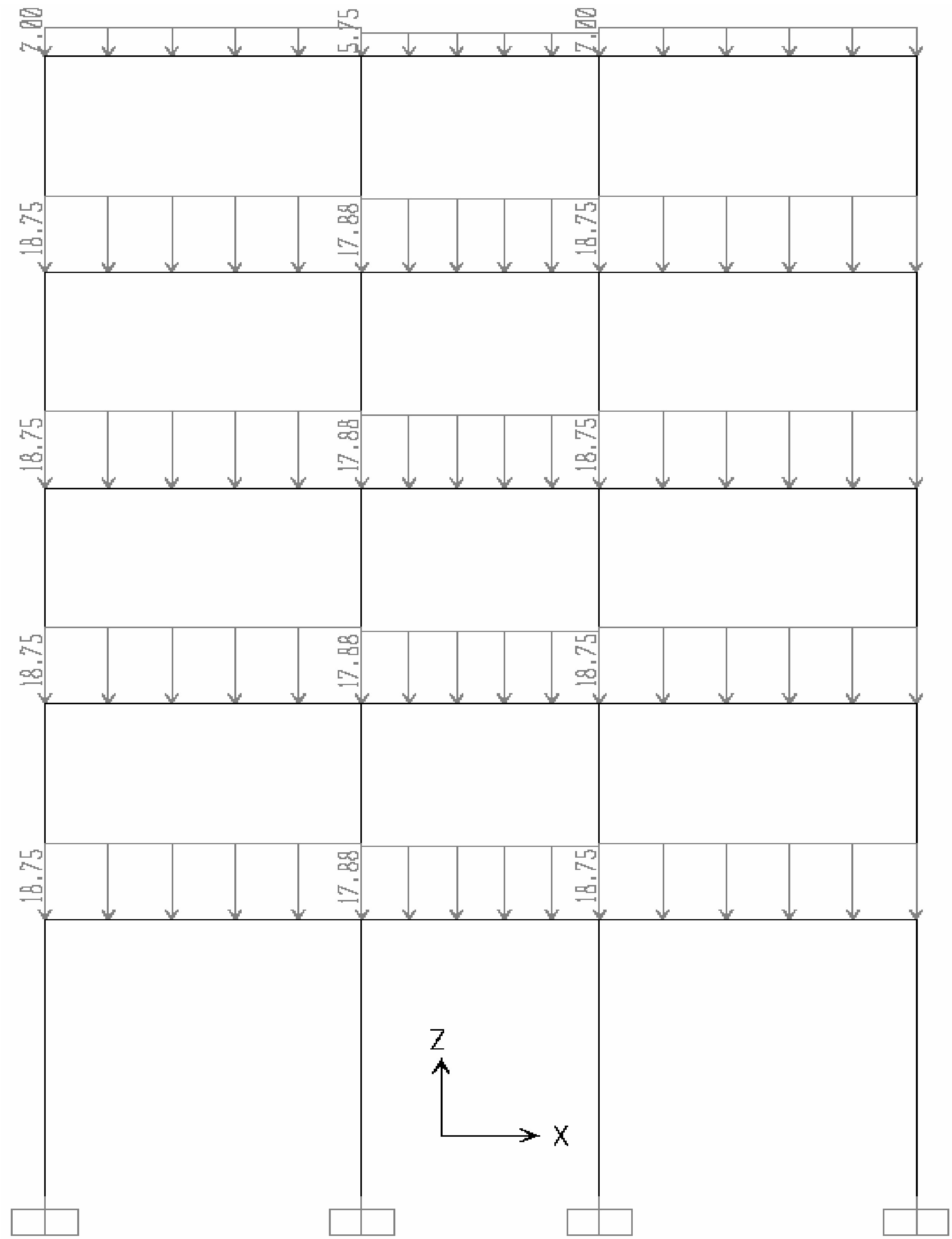
= 271.38 KN.

Table 1 Calculation of earthquake load in X direction at each floor level:

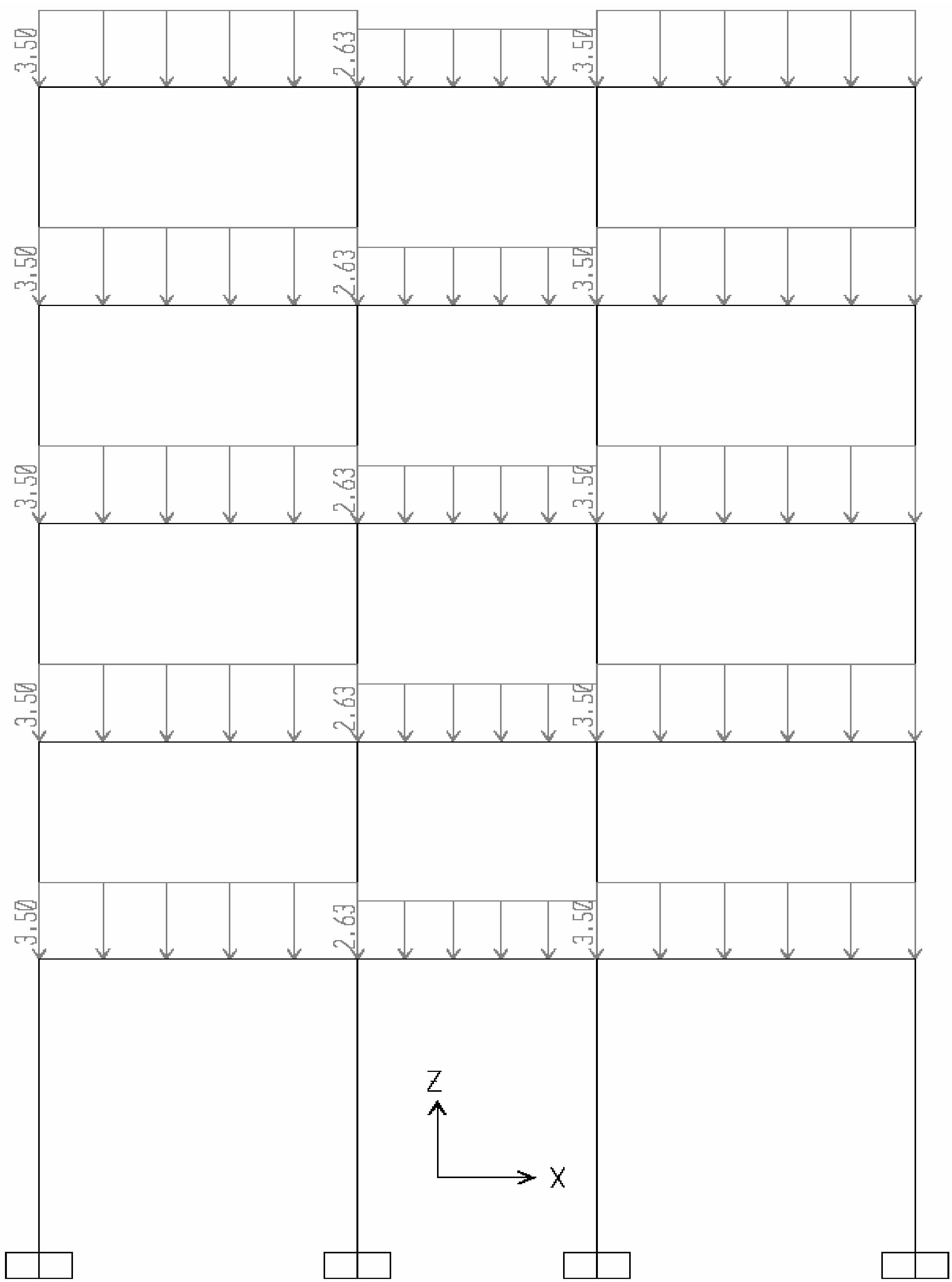
Level	W_i in KN	h_i in m	$W_i h_i^2$ in KN.m^2	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	V_b in KN	Q in KN
Roof	2734.363	18.5	935835.6	0.355997	271.38	96.61
4th floor	3833.425	15	862520.6	0.328108		89.04
3rd floor	3833.425	11.5	506970.5	0.192855		52.34
2nd floor	3833.425	8	245339.2	0.093328		25.33
1st floor	3857.05	4.5	78105.26	0.029712		8.06
GF		0	0			
	∑=18091.69		∑=2628771			

Table 2 Calculation of earthquake load in Y direction at each floor level:

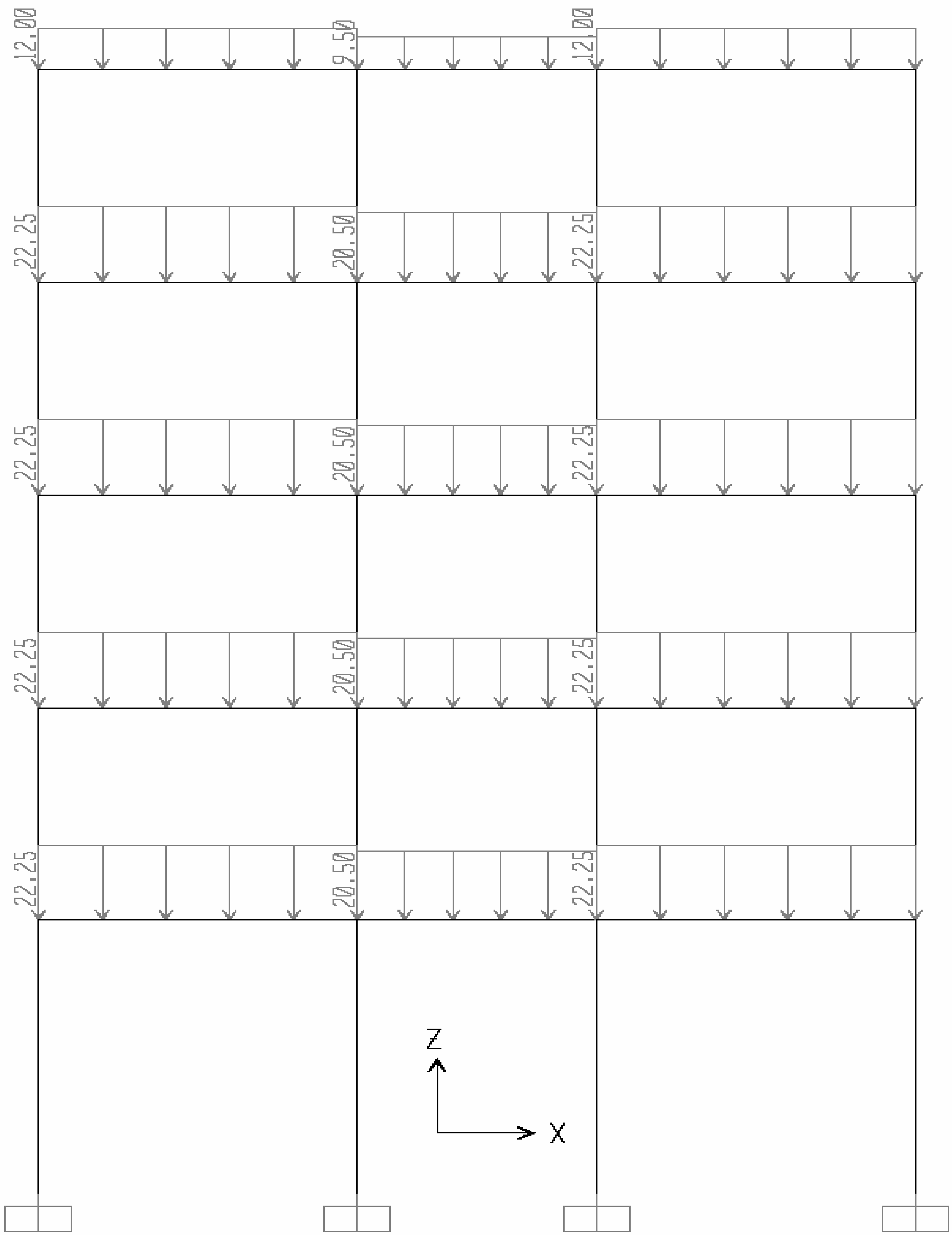
Level	W_i in KN	h_i in m	$W_i h_i^2$ in KN.m^2	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	V_b in KN	Q in KN
Roof	2734.363	18.5	935835.6	0.355997	407.06	144.91
4th floor	3833.425	15	862520.6	0.328108		133.56
3rd floor	3833.425	11.5	506970.5	0.192855		78.51
2nd floor	3833.425	8	245339.2	0.093328		37.99
1st floor	3857.05	4.5	78105.26	0.029712		12.09
GF		0	0			
	∑=18091.69		∑=2628771			



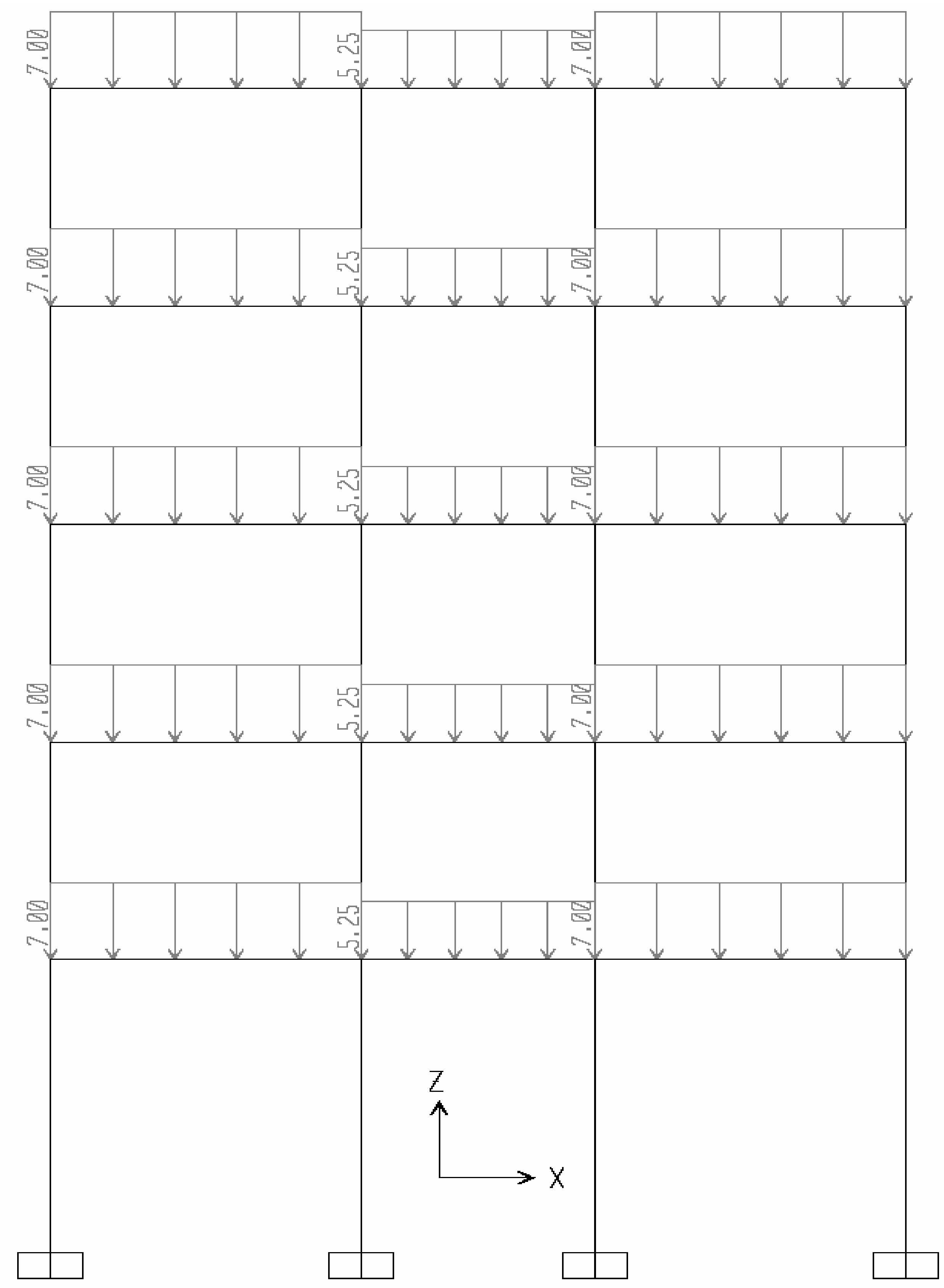
Dead load on end frame in XZ plane
Figure-8



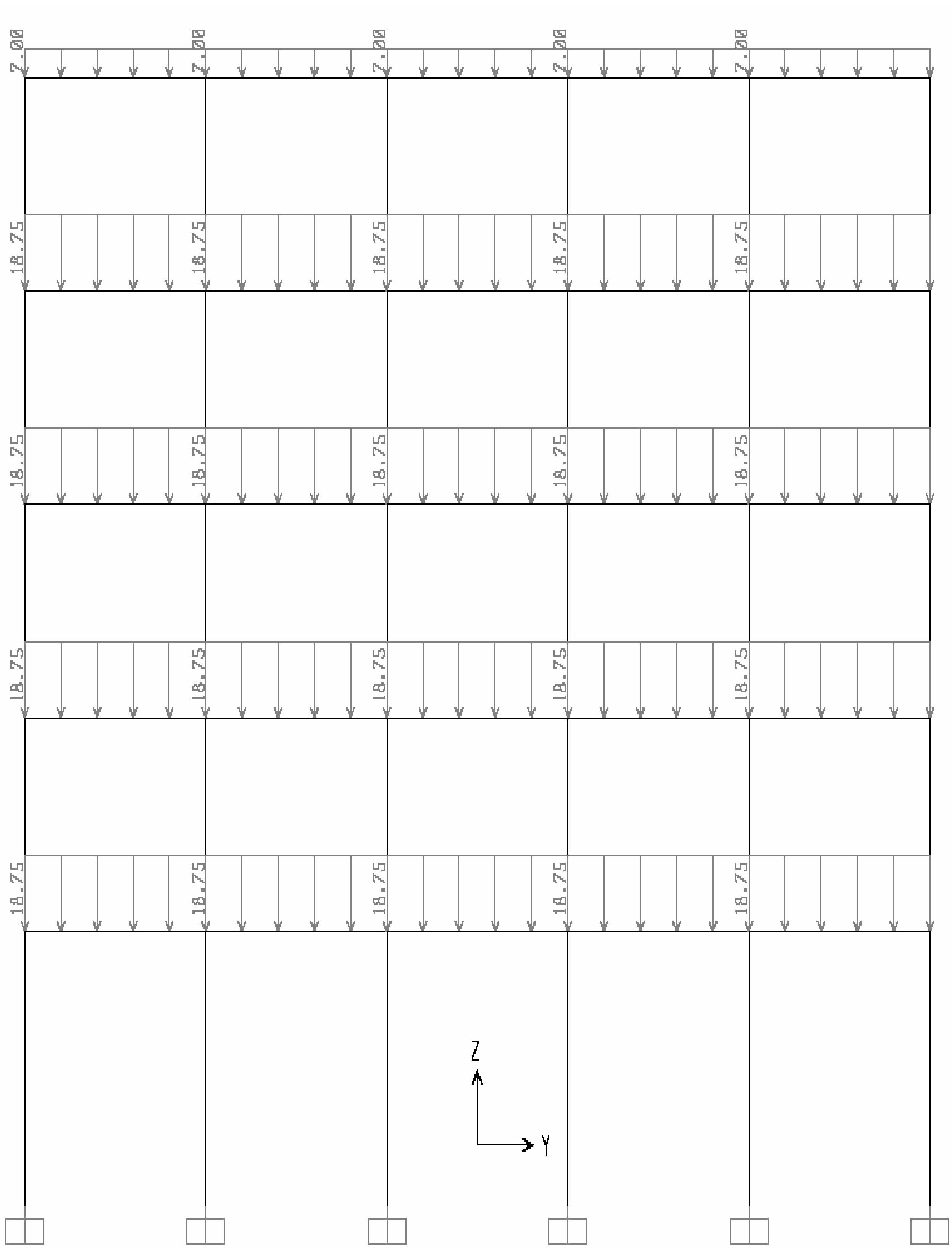
Live load on end frame in XZ plane
Figure-9



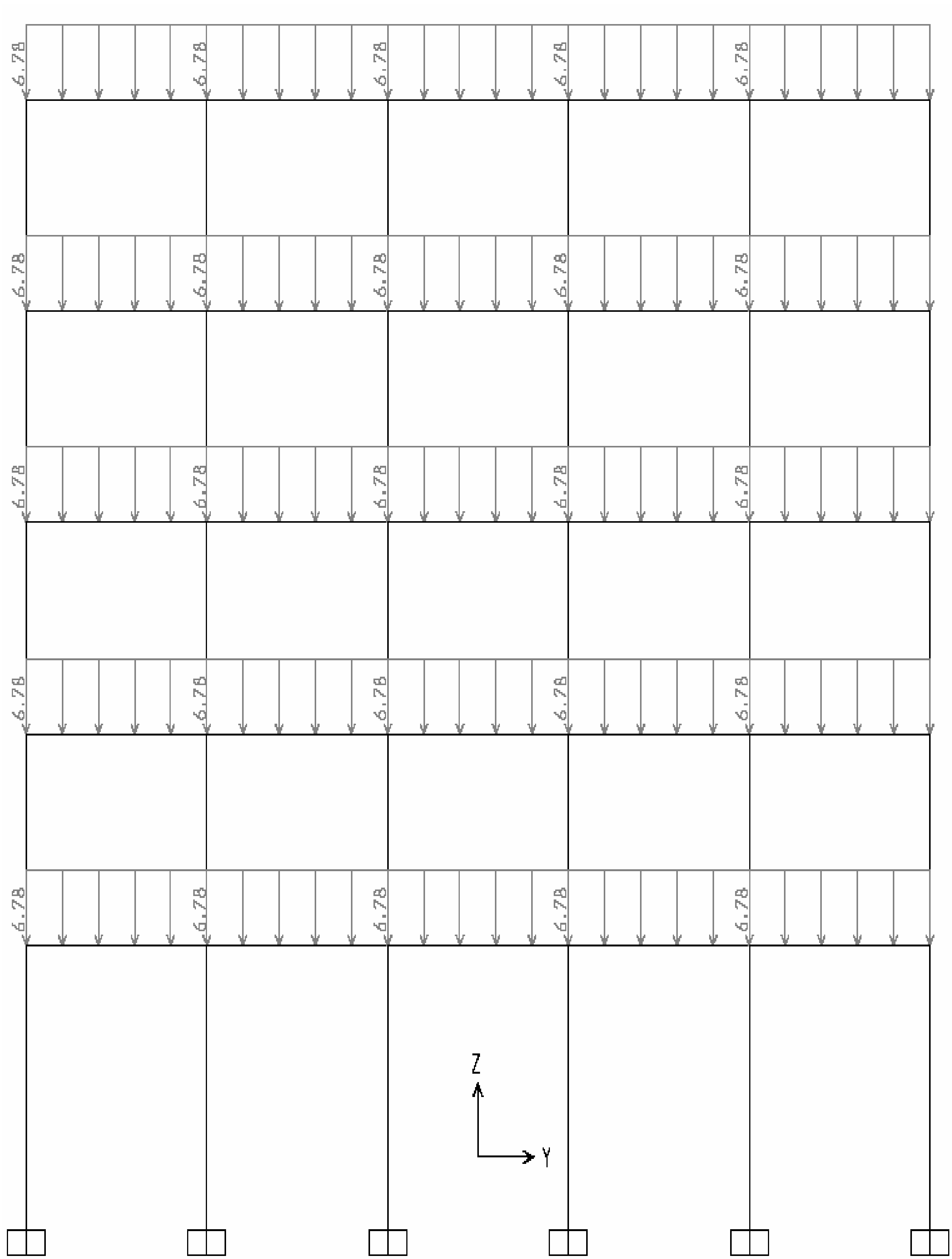
Dead load on intermediate frame in XZ Plane
Figure-10



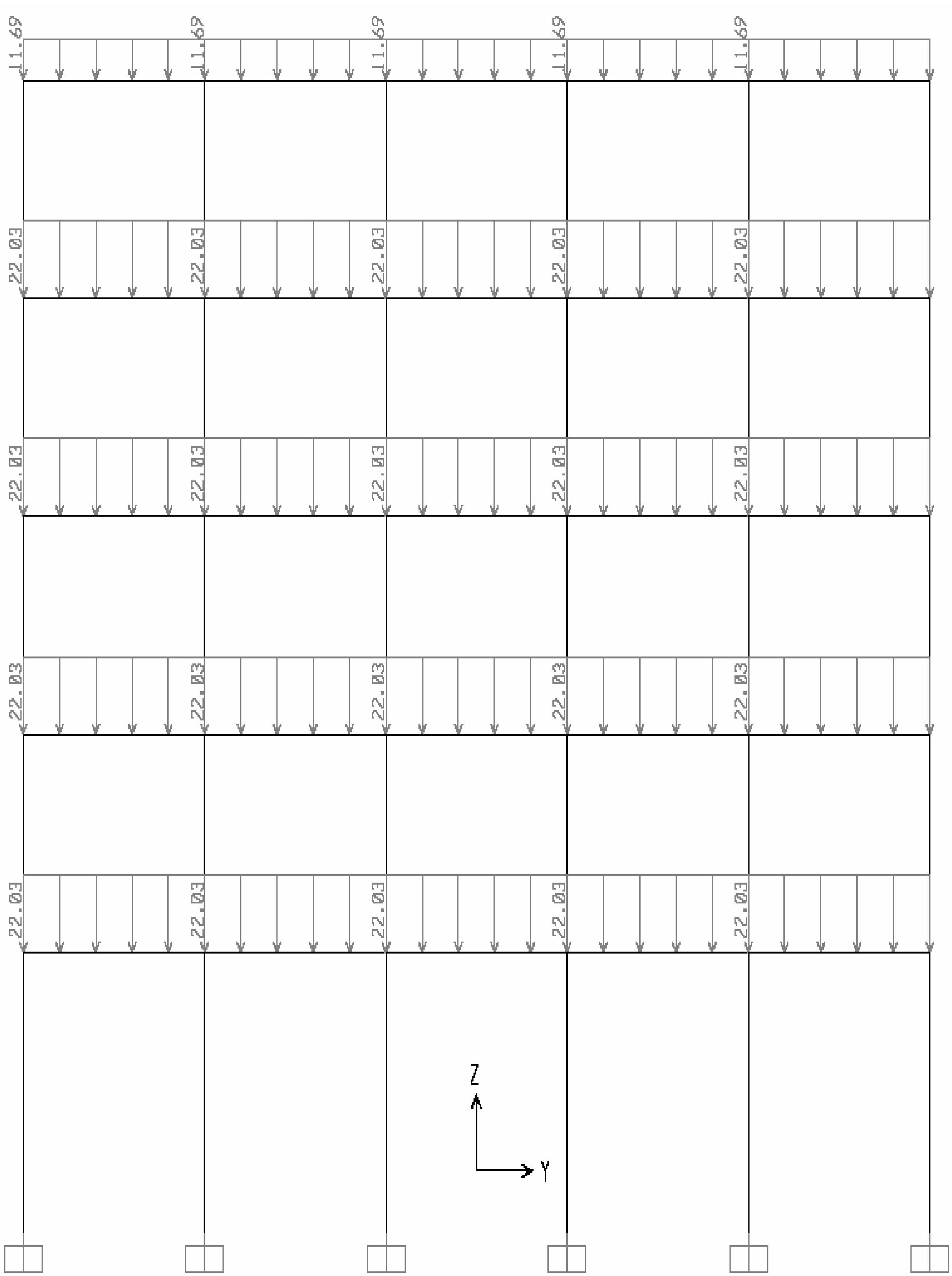
Live load on intermediate frame in XZ plane
Figure-11



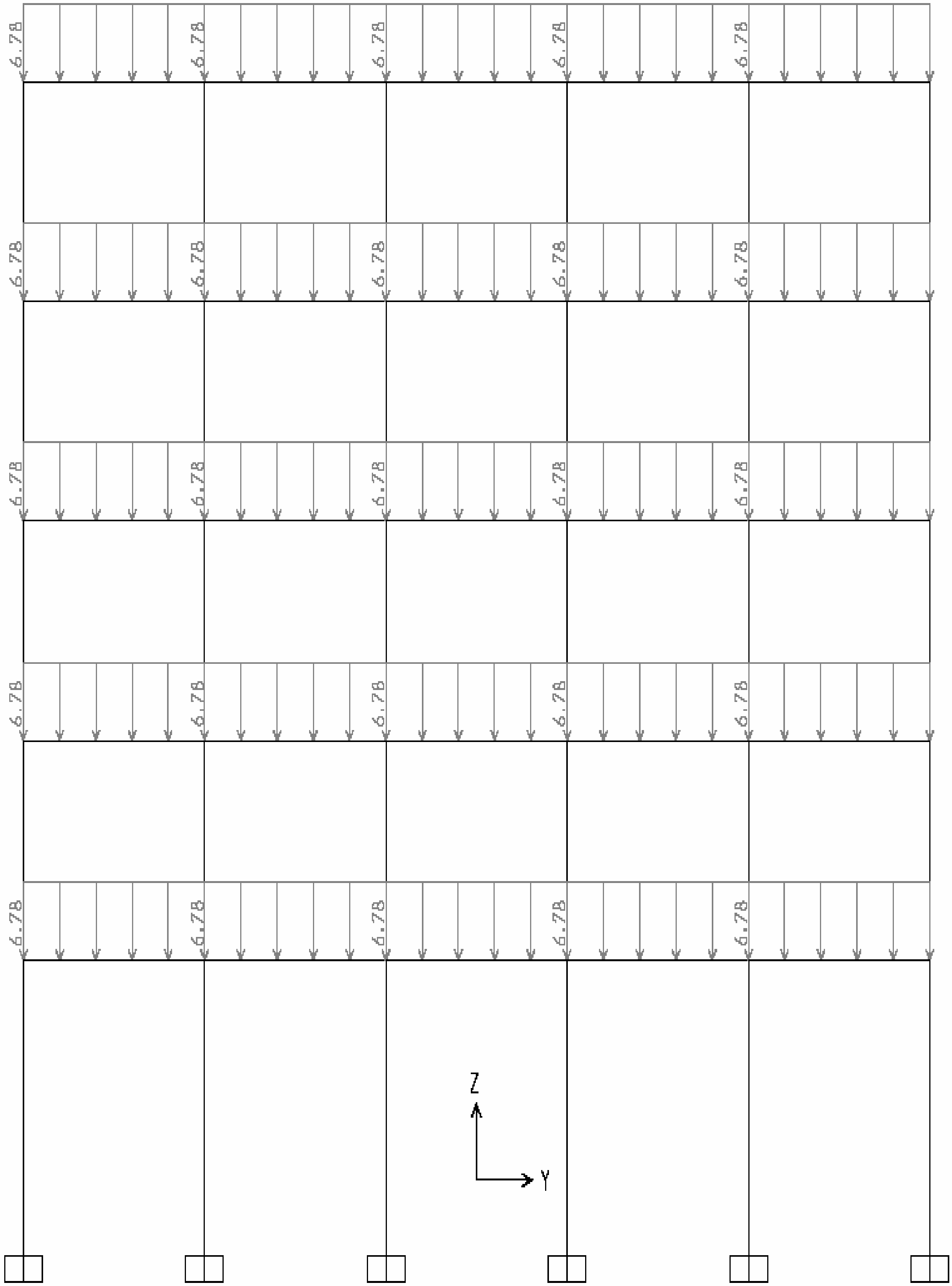
Dead load on end frame in YZ plane
Figure-12



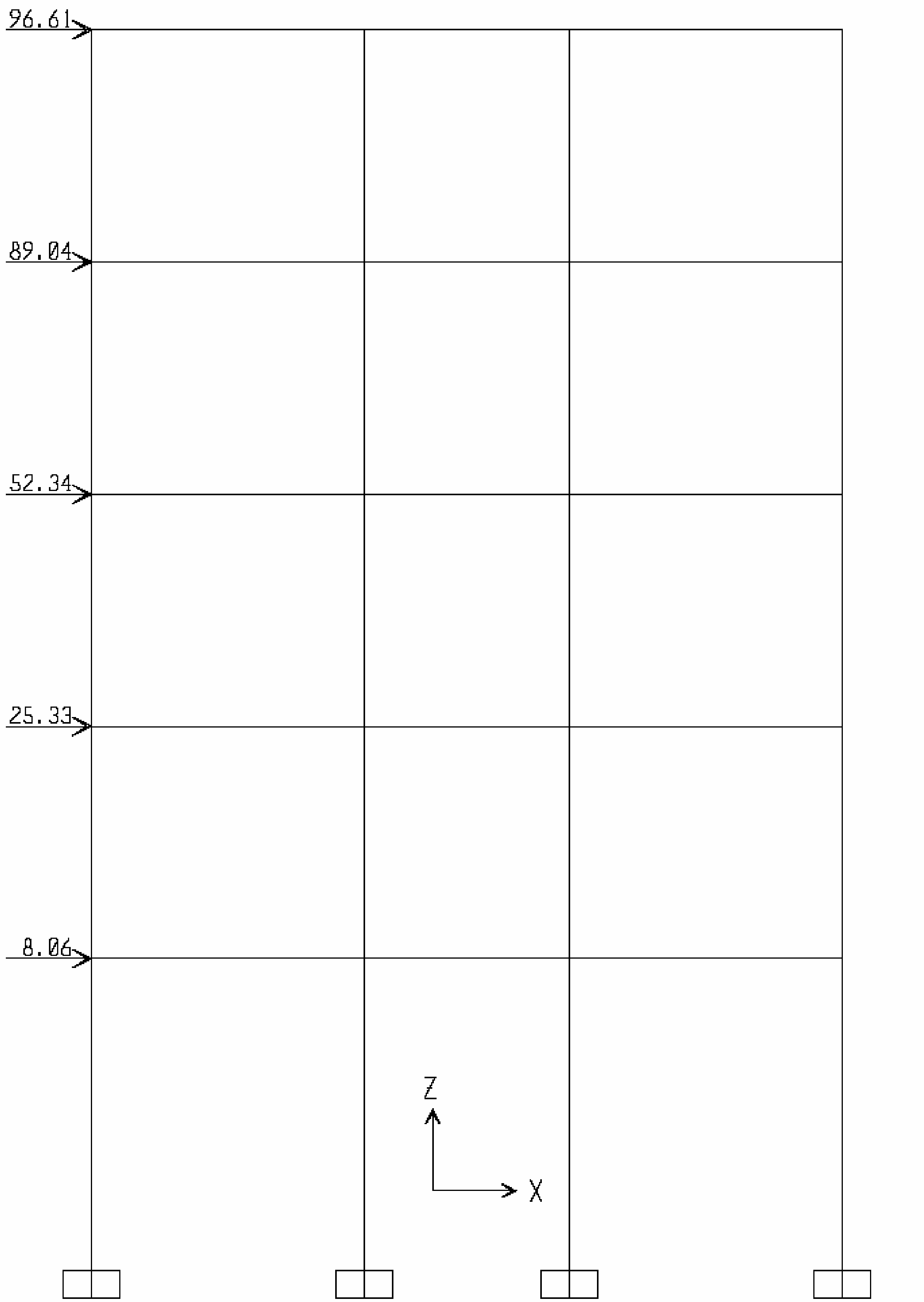
Live load on end frame in YZ plane
Figure-13



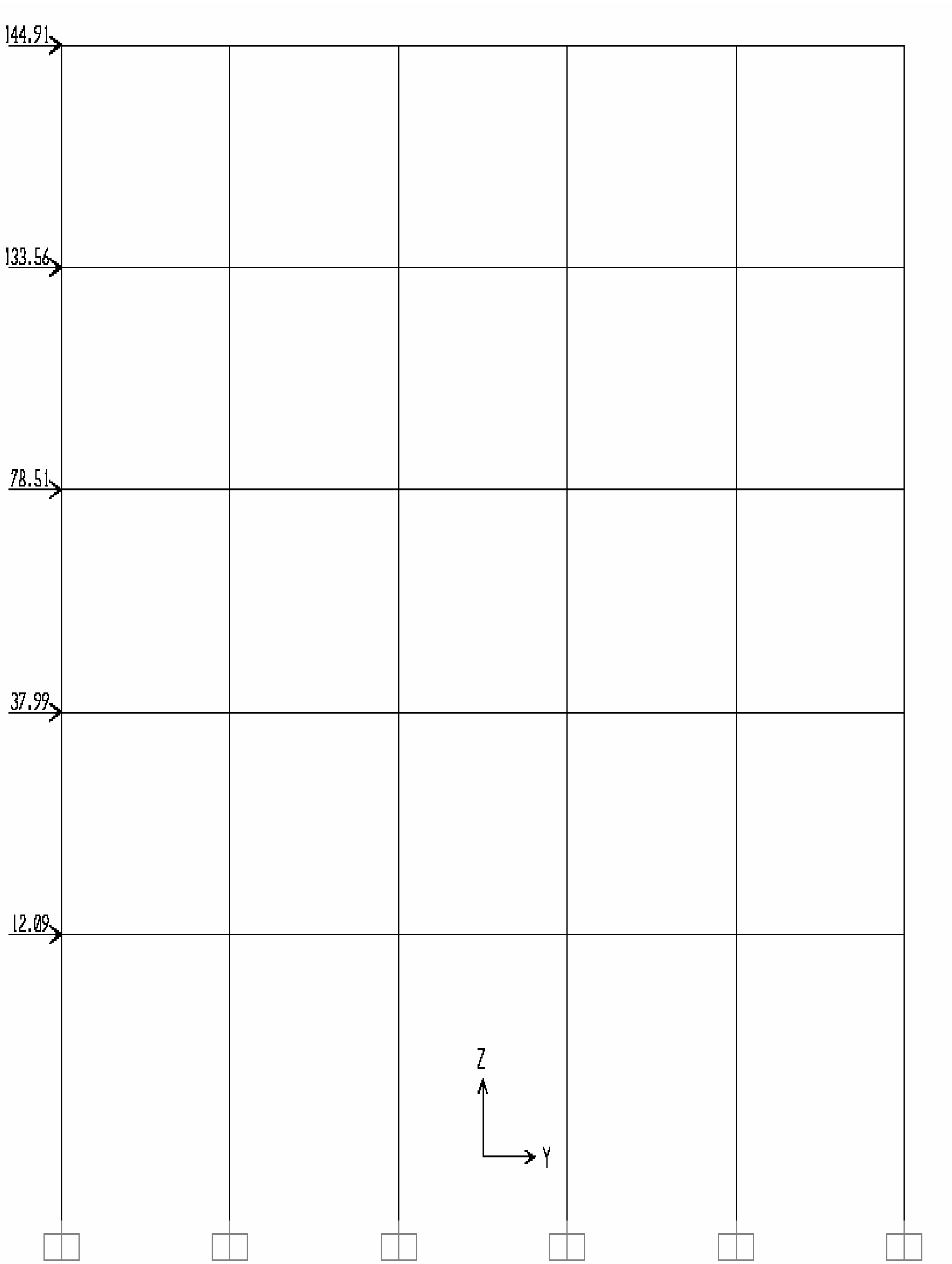
Dead load on intermediate frame in YZ plane
Figure-14



Live load on intermediate frame in YZ plane
Figure-15



Earthquake load on end frame & intermediate frame in X Direction
Figure-16



Earthquake load on end frame & intermediate in Y direction
Figure-17

Using the above loading data, analysis of the frame is carried out with all the load combinations as per IS 1893(Part 1):2002. The maximum moments and forces for the beams and columns for all the load combinations for each member are considered for the design.

The different load combinations are:

1. 1.5(DL+IL)
2. 1.2(DL+IL+EL)
3. 1.2(DL+IL-EL)
4. 1.5(DL+EL)
5. 1.5(DL-EL)
6. 0.9DL+1.5EL
7. 0.9DL-1.5EL

The analysis of the frame is done by SAP-2000. The results are given in appendix 2 and the maximum values in all combinations are considered.

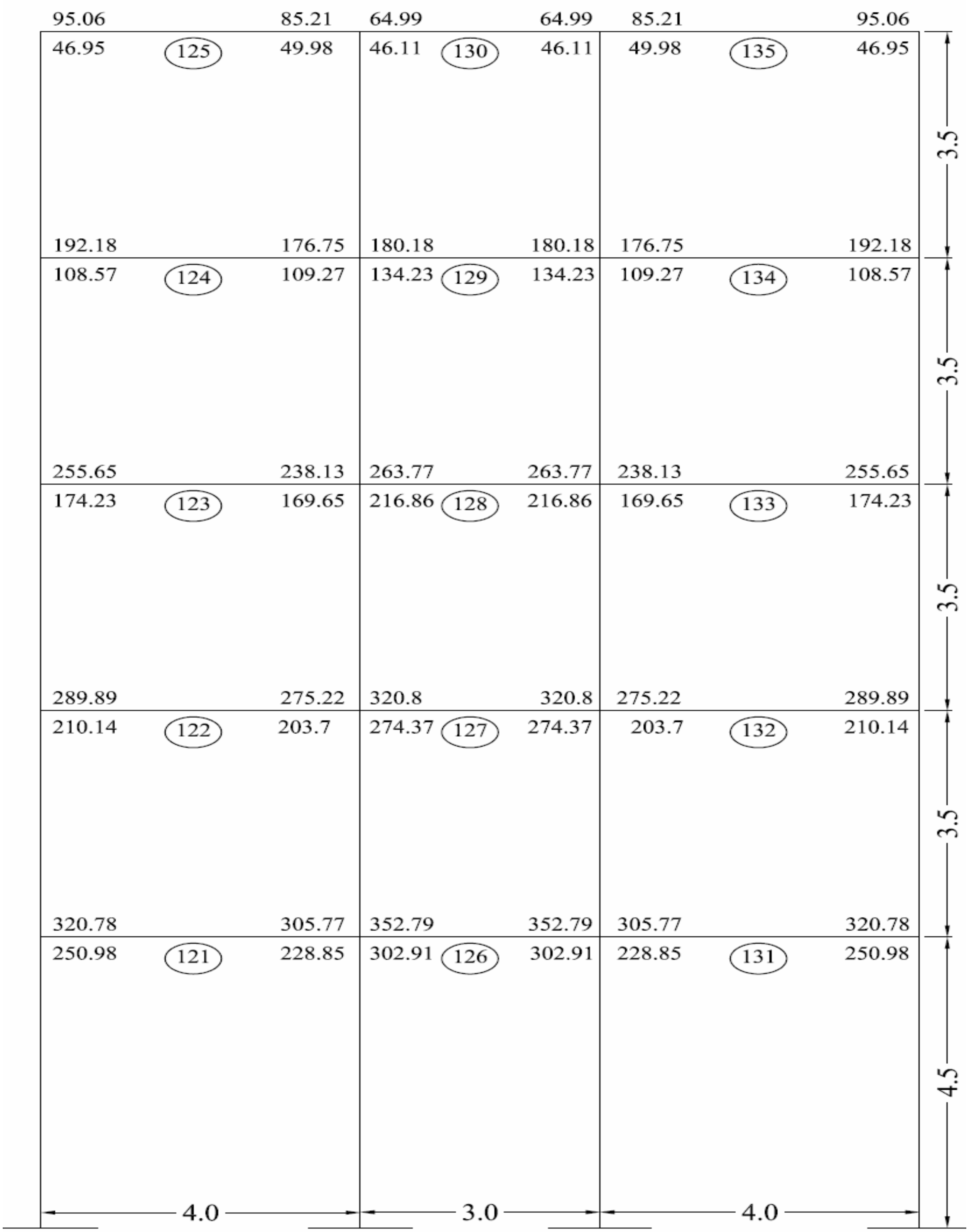
CHAPTER -5

Capacity Based design of 3D-RC Frame

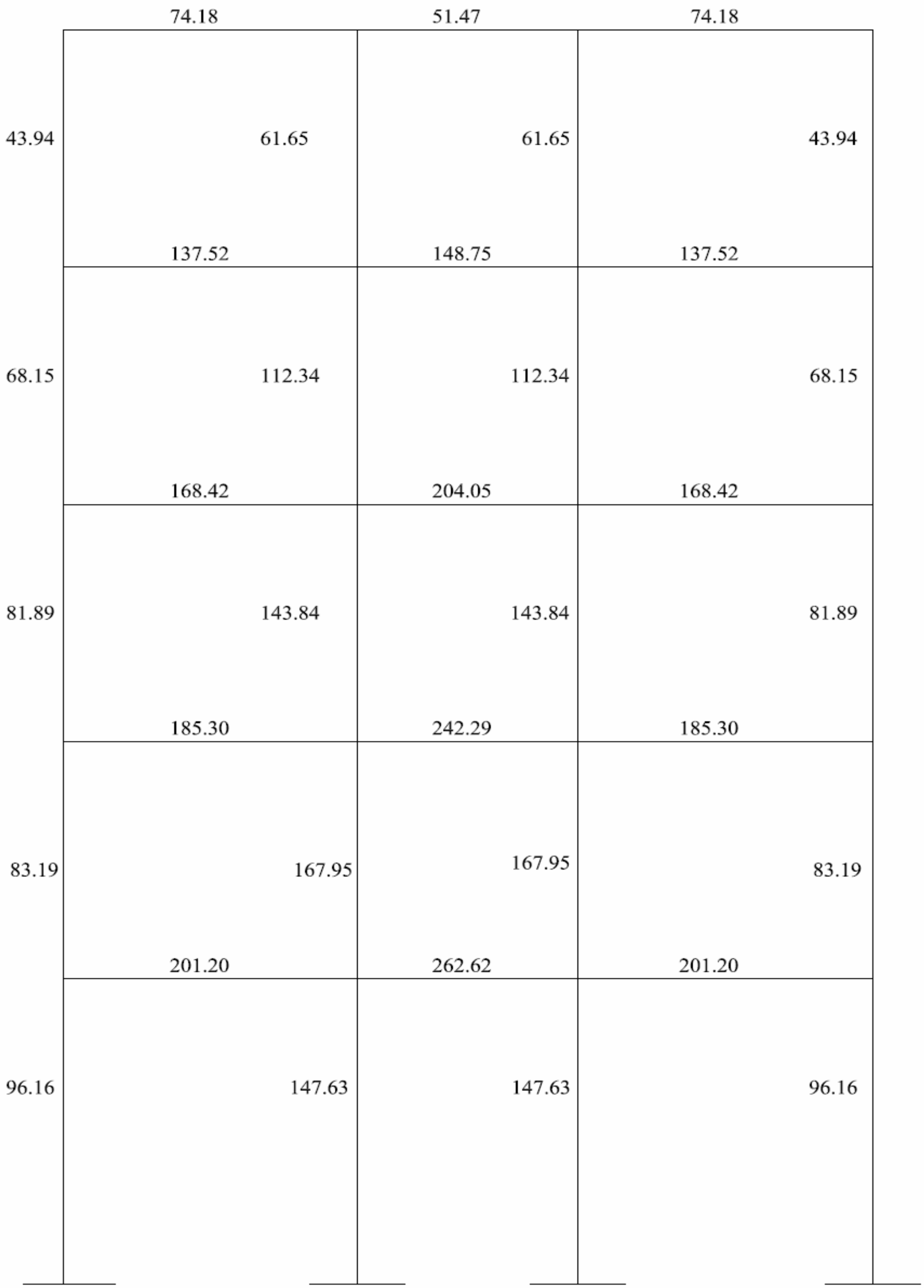
Chapter - 5.1

Design of End Frame in XZ Plane

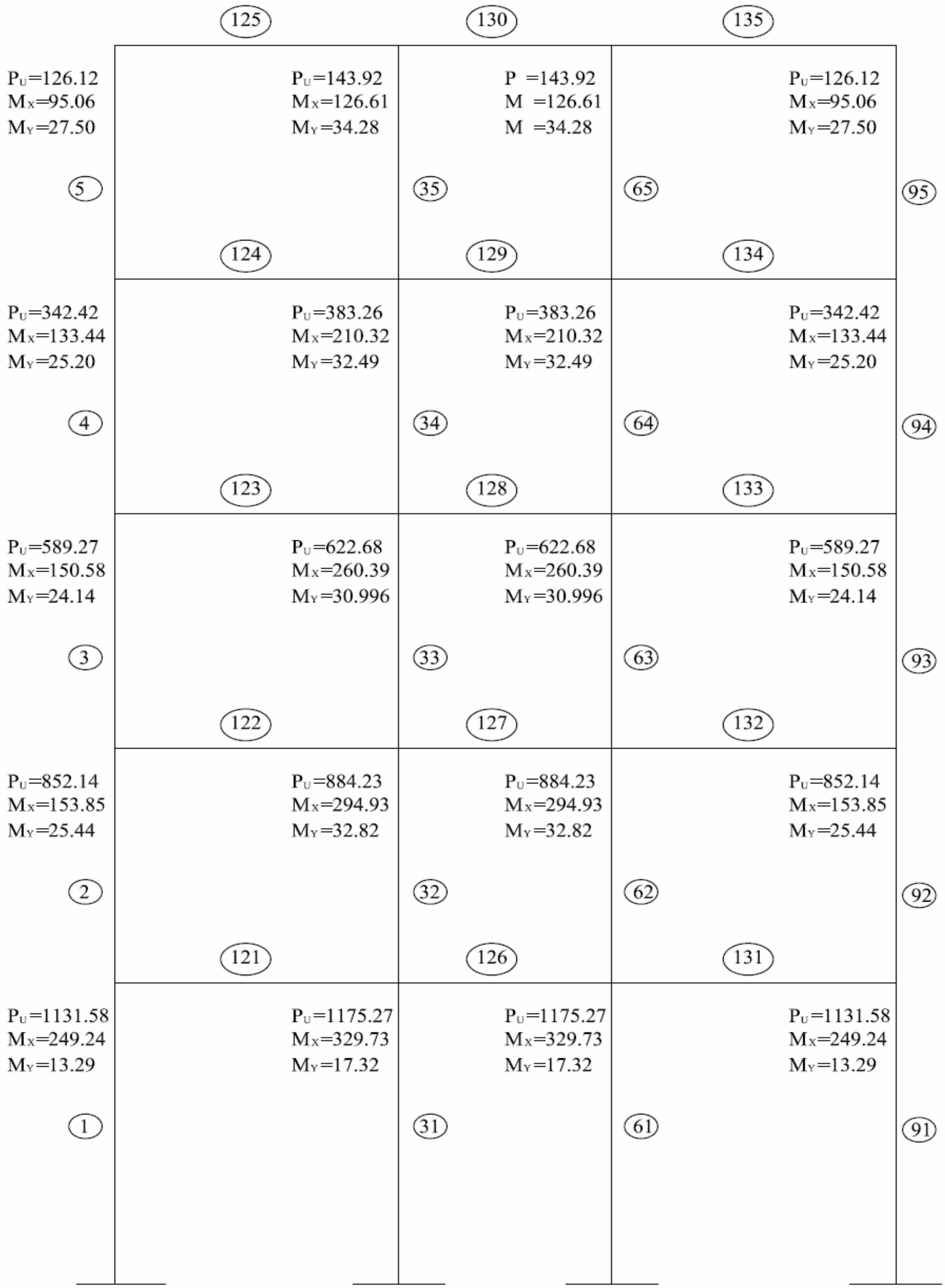
Design of End Frame in XZ Plane:



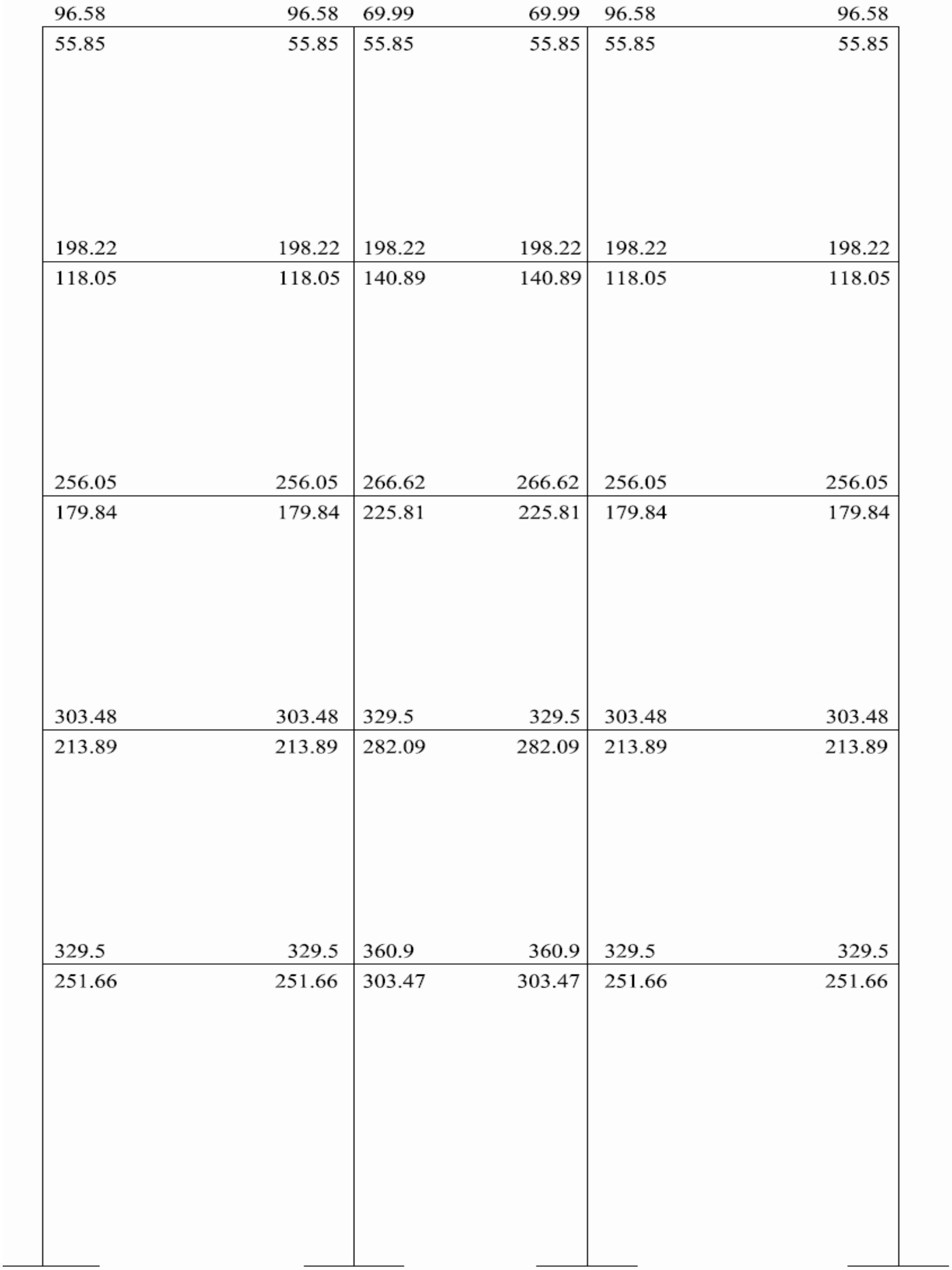
Maximum Sagging (Below) & Hogging (Above) of Beams.
Figure-18



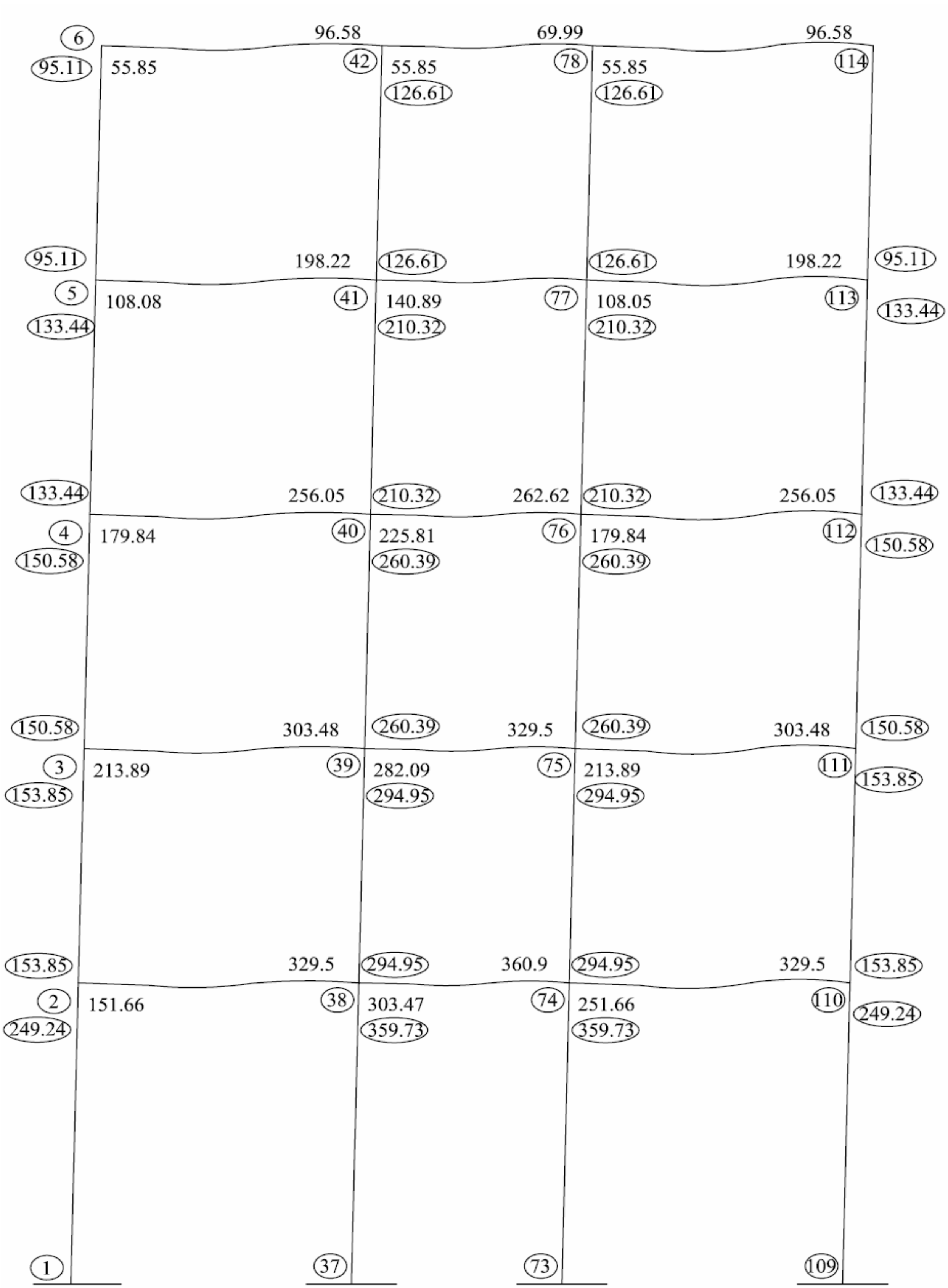
Maximum Shear Force
Figure-19



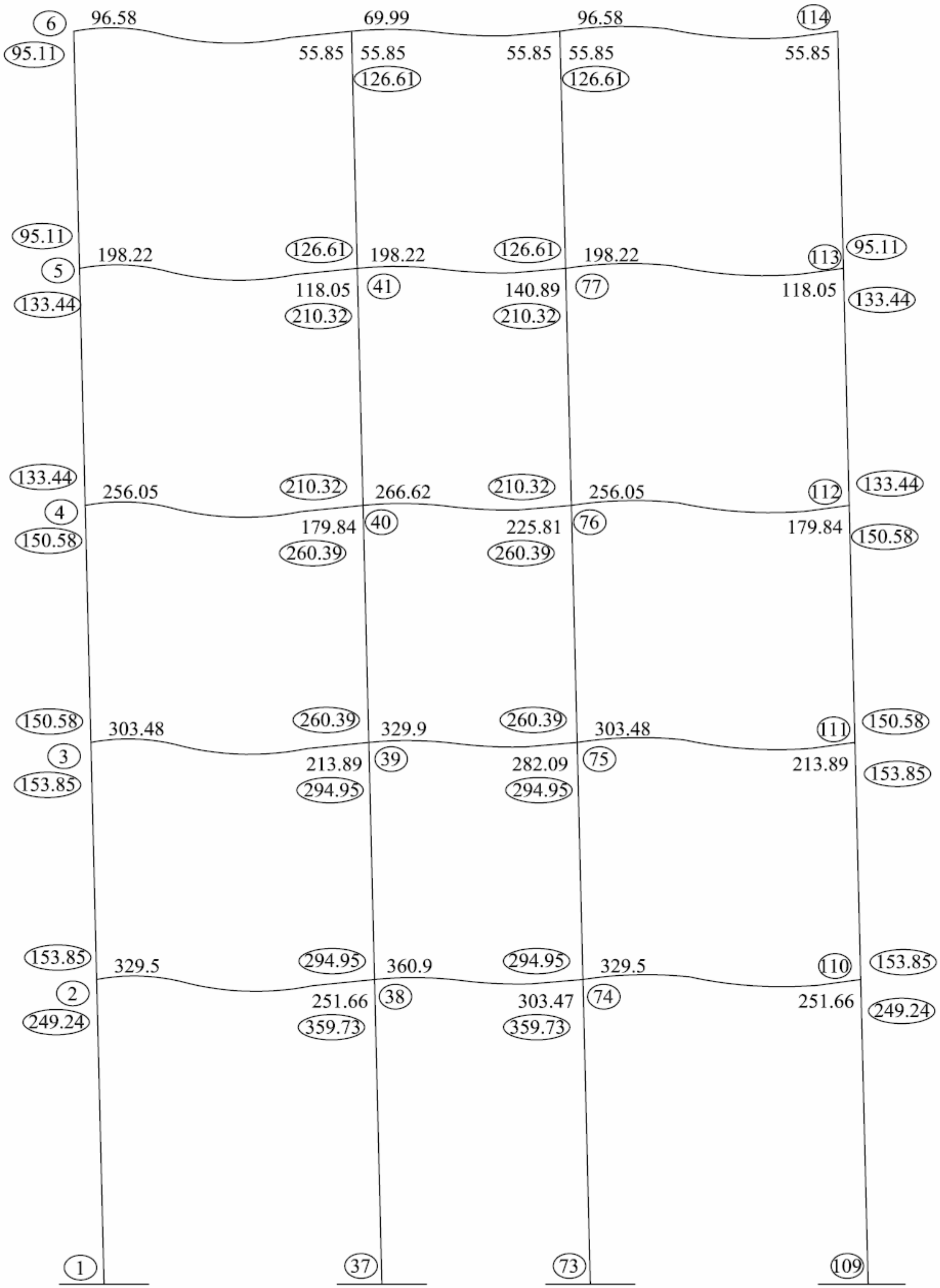
Axial Force & Biaxial Moments of Columns
Figure-20



Moment of Resistance of Beams as per provided Reinforcement
Figure-21



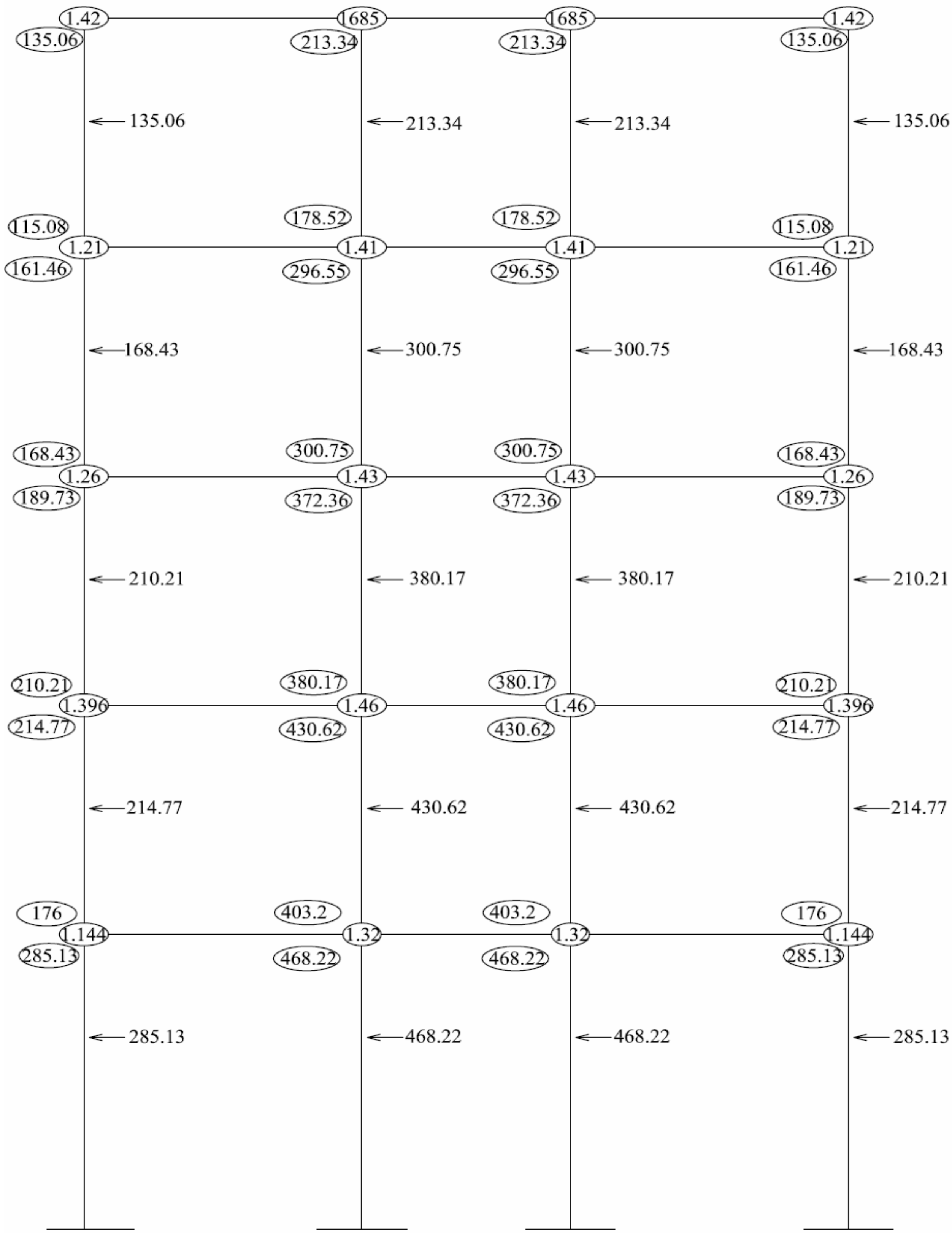
Seismic Direction 1
Figure-22



Seismic Direction 2
Figure-23

Table 3 Determination of moment magnification factor of columns of end frame in XZ plane at all joints:

Joint No.	Seismic Direction	Sum of resisting moments of top & bottom of columns at joint. **1**	Sum of resisting moments of left & right beams at joint with an over strength factor 1.4. **2**	Check for **1≥2**	Moment Magnification factor **2÷1**
6	1	0+95.11 = 95.11	1.4(0+55.85) = 78.85	Ok	1.0
	2	0+95.11 = 95.11	1.4(0+96.58) = 135.21	Not Ok	1.42
42	1	0+126.61 = 126.61	1.4(96.58+55.85+) = 213.4	Not Ok	1.685
	2	0+126.61 = 126.61	1.4(69.99+55.85) = 176.18	Not Ok	1.39
5	1	95.11+133.44 = 228.55	1.4(0+118.05) = 165.27	Ok	1.0
	2	95.11+133.44 = 228.55	1.4(0+198.22) = 277.08	Not Ok	1.21
41	1	126.61+210.32 = 336.93	1.4(198.22+140.89) = 474.75	Not Ok	1.41
	2	126.61+210.32 = 336.93	1.4(118.05+198.22) = 442.78	Not Ok	1.31
4	1	133.44+150.58 = 284.02	1.4(0+179.84) = 251.78	Ok	1.0
	2	133.44+150.58 = 284.02	1.4(0+256.05) = 358.47	Not Ok	1.26
40	1	210.32+260.39 = 470.71	1.4(256.05+225.81) = 674.6	Not Ok	1.43
	2	210.32+260.39 = 470.71	1.4(266.62+179.84) = 625.04	Not Ok	1.328
3	1	150.58+153.85 = 304.43	1.4(0+213.89) = 299.45	Ok	1.0
	2	150.58+153.85 = 304.43	1.4(0+303.48) = 424.87	Not Ok	1.396
39	1	260.39+294.95 = 555.34	1.4(303.48+282.09) = 819.9	Not Ok	1.48
	2	260.39+294.95 = 555.34	1.4(213.89+329.5) = 760.75	Not Ok	1.36
2	1	153.85+249.24 = 403.09	1.4(0+251.66) = 352.32	Ok	1.0
	2	153.85+249.24 = 403.09	1.4(0+329.5) = 461.3	Not Ok	1.144
38	1	294.95+329.73 = 624.68	1.4(329.5+303.47) = 886.16	Not Ok	1.42
	2	294.95+329.73 = 624.68	1.4(251.66+360.9) = 857.58	Not Ok	1.37



Revision of Column Moments According to Capacity Based Design by Moment Magnification Factor
Figure-24

Table 4 Revised design capacity of columns of end frame in XZ plane with earthquake force in X direction:

Storey No.	Column No.	Column Size mm×mm	P_{uz} In KN	M_{ux} In KN	M_{uy} In KN	% of Steel	Interaction Ratio
5	5, 95	400×500	126.12	135.06	27.5	1.1	0.985
	35, 65	400×550	143.92	213.34	34.28	1.125	0.998
4	4, 94	400×500	342.42	168.43	25.2	1.15	0.995
	34, 64	400×550	383.26	300.75	32.49	1.625	0.994
3	3, 93	400×500	589.27	210.21	24.14	1.225	0.99
	33,63	400×550	622.68	380.17	30.996	1.875	0.999
2	2, 92	400×500	852.14	214.77	25.44	1.45	0.988
	32, 62	400×550	885.6	430.125	38.82	2.4	0.991
1	1, 91	400×500	1131.58	285.13	28.02	2.15	0.987
	31, 61	400×550	1174.85	468.22	31.05	2.85	0.989

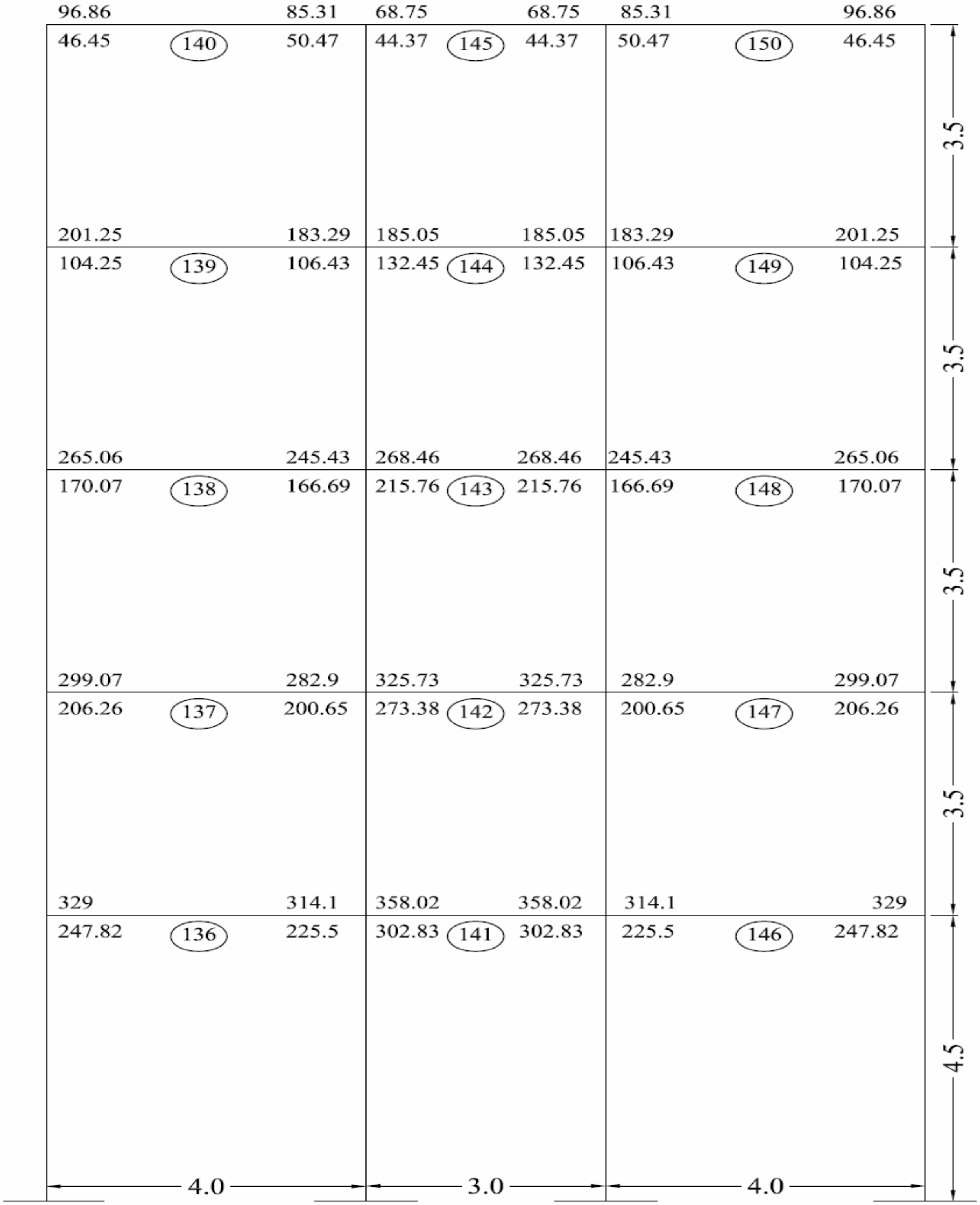
Table 5 Capacity based shear & shear reinforcement in beams of end frame in XZ plane:

Beam No.	Shear in Seismic direction 1 In KN	Shear in Seismic direction 2 In KN	Maximum shear force In KN	Shear force from analysis In KN	Design shear force, V_u In KN	Shear Reinforcement Provided
125,135	-30.73 75.97	75.97 -30.73	75.97	74.18	75.97	8Φ two legged stirrup @ 100mm c/c.
130	-44.48 72.96	72.96 -44.48	72.96	51.57	72.96	8Φ two legged stirrup @ 100mm c/c.
124,134	-55.07 166.31	166.31 -55.07	166.01	137.52	166.01	8Φ two legged stirrup @ 100mm c/c.
129	-118.58 197.91	197.91 -118.58	197.91	148.75	197.91	8Φ two legged stirrup @ 100mm c/c.
123,133	-96.94 208.18	208.18 -96.94	208.18	168.42	208.18	8Φ two legged stirrup @ 100mm c/c.
128	-189.9 269.2	269.2 -189.9	269.2	204.05	269.2	8Φ two legged stirrup @ 80mm c/c.
122,132	-125.46 236.69	236.69 -125.46	236.69	185.3	236.69	8Φ two legged stirrup @ 65mm c/c.
127	-243.11 312.44	312.44 -243.11	312.44	242.29	312.44	8Φ two legged stirrup @ 65mm c/c.
121,131	-147.8 259.03	259.03 -147.8	259.03	201.2	259.03	8Φ two legged stirrup @ 85mm c/c.
126	-273.37 331.04	331.04 -273.37	331.04	262.62	331.04	8Φ two legged stirrup @ 60mm c/c.

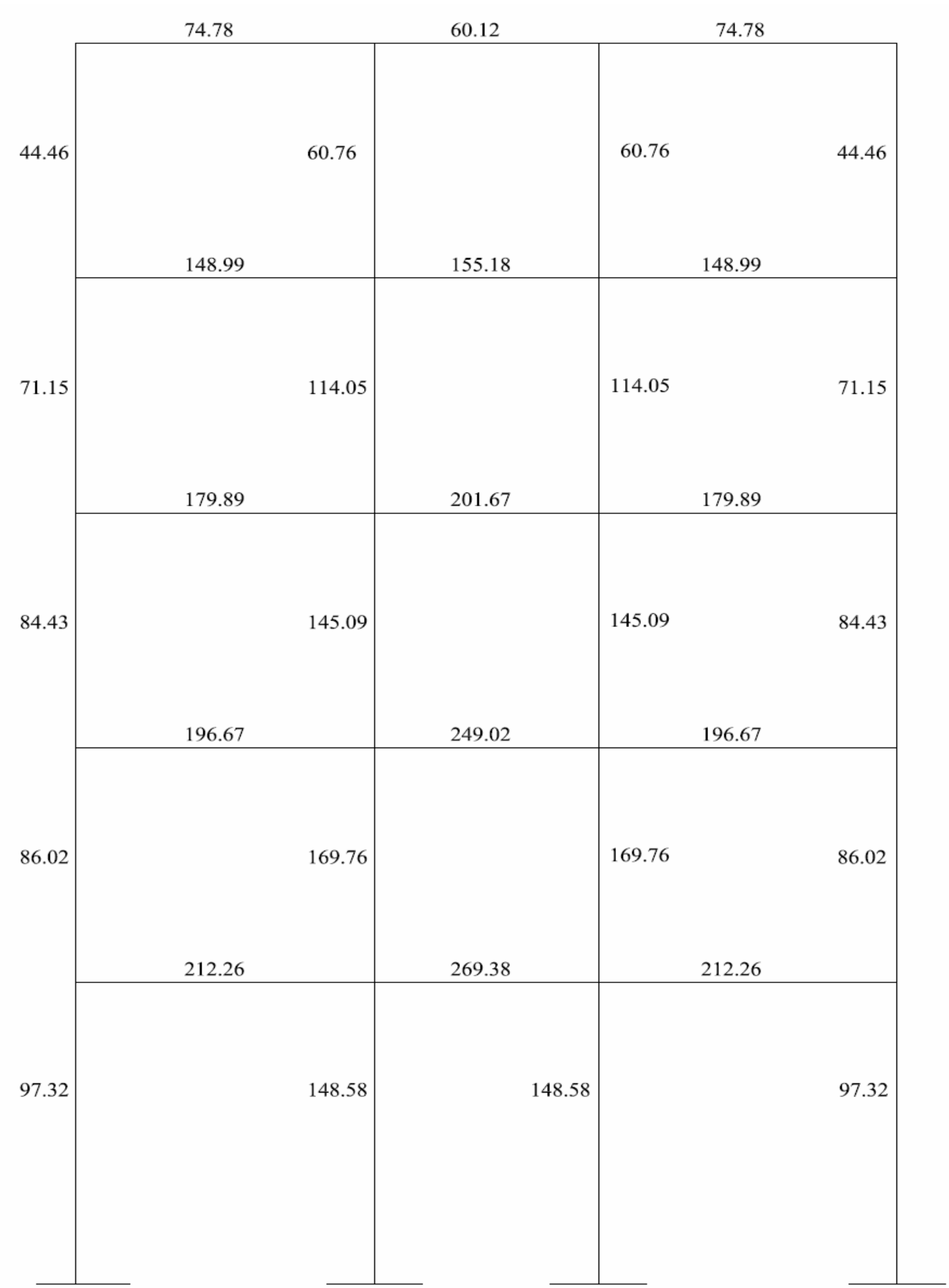
Chapter -5.2

Design of Intermediate Frame in XZ Plane

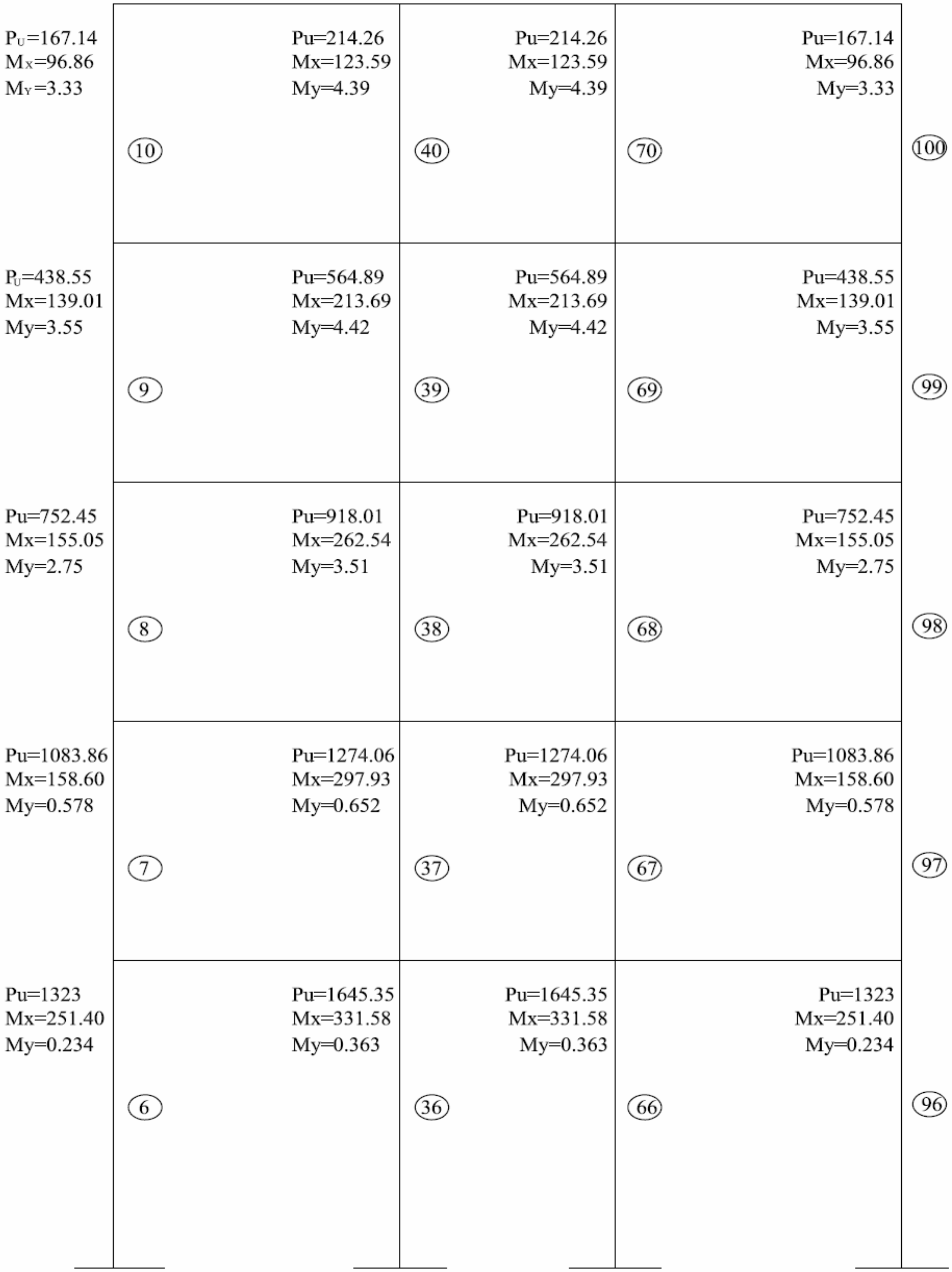
Design of Intermediate Frame in XZ Plane:



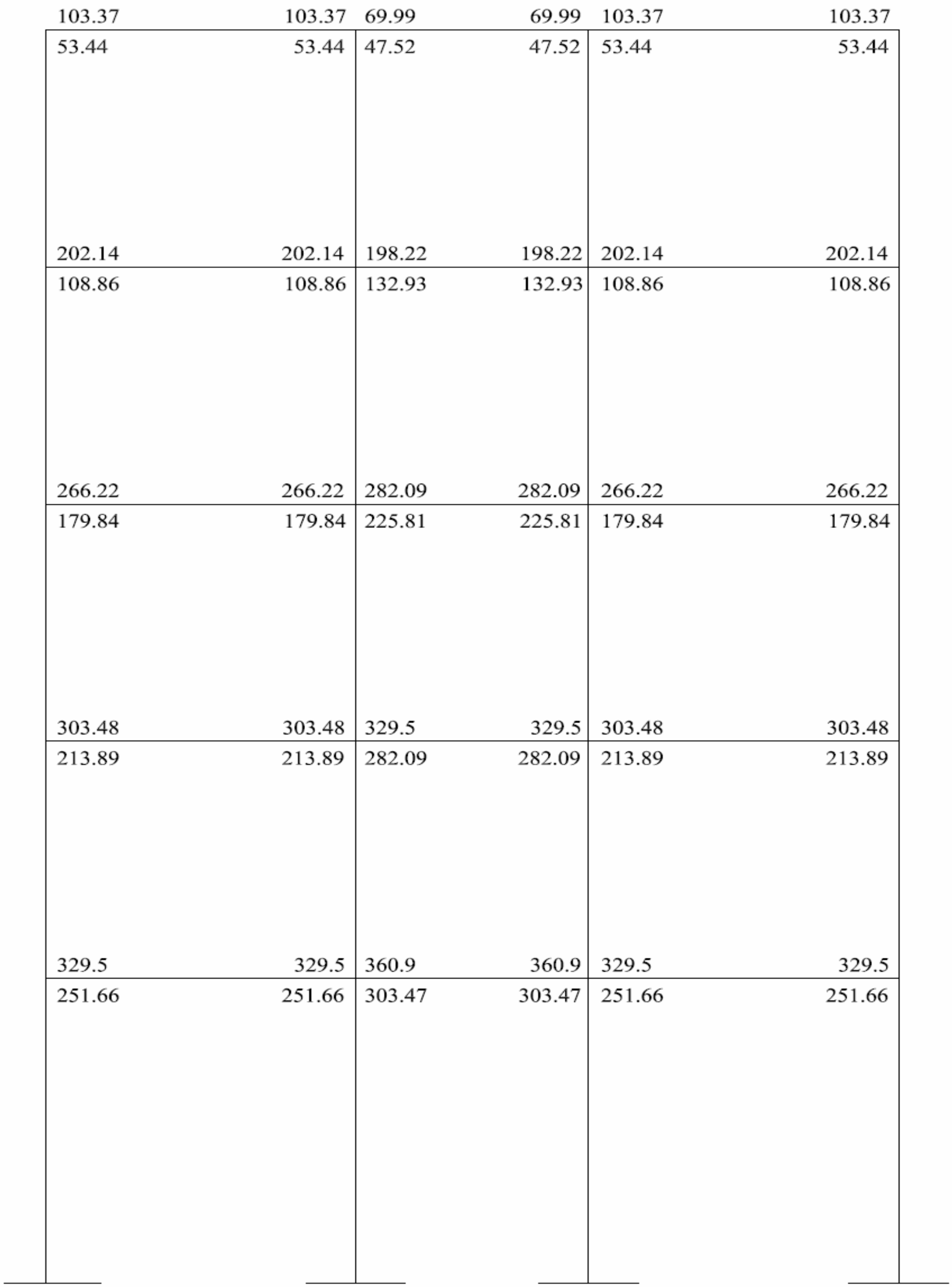
Maximum Sagging (Below) & Hogging (Above) Moments of Beams
Figure-25



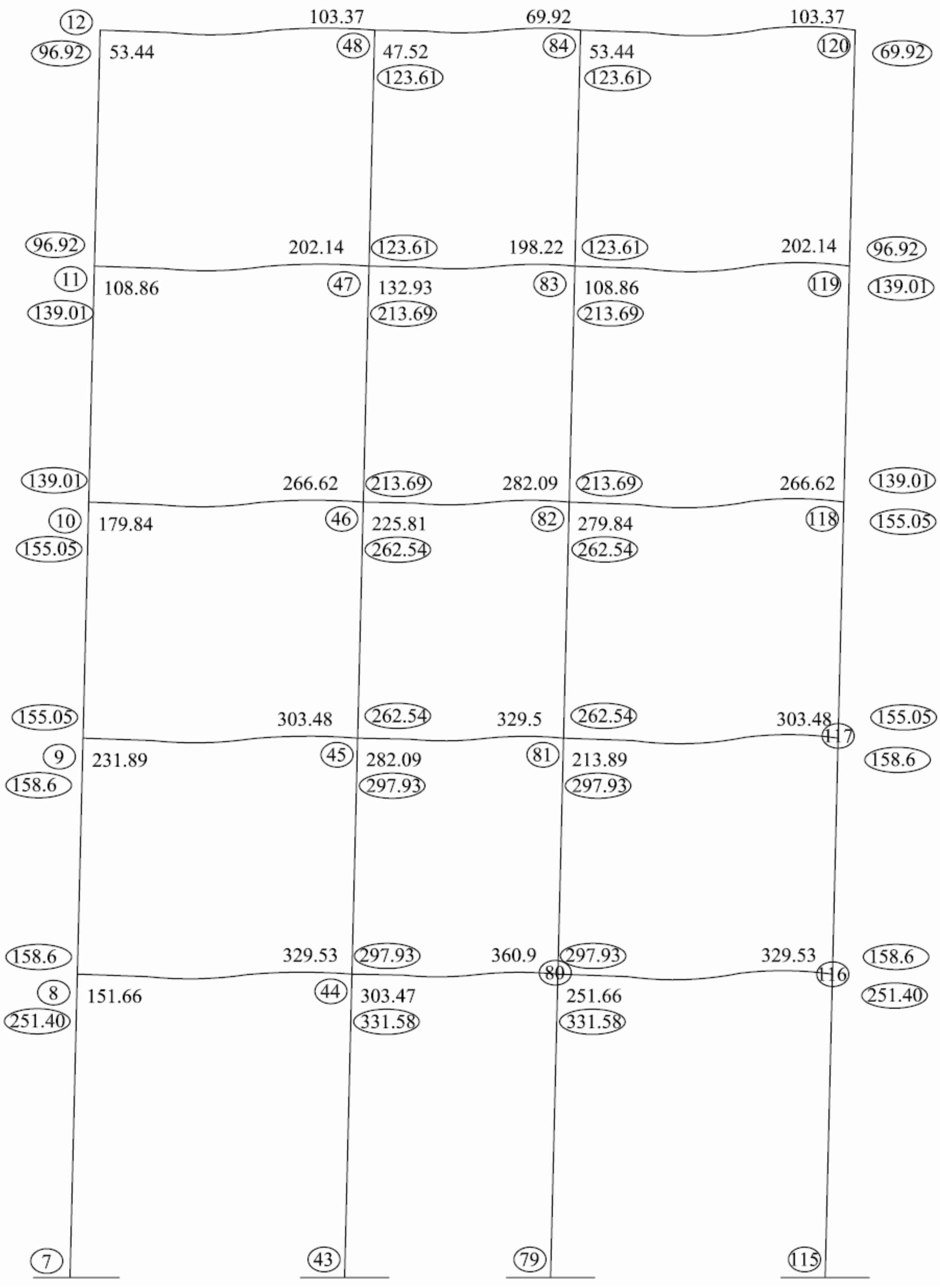
Maximum Shear Force
Figure-26



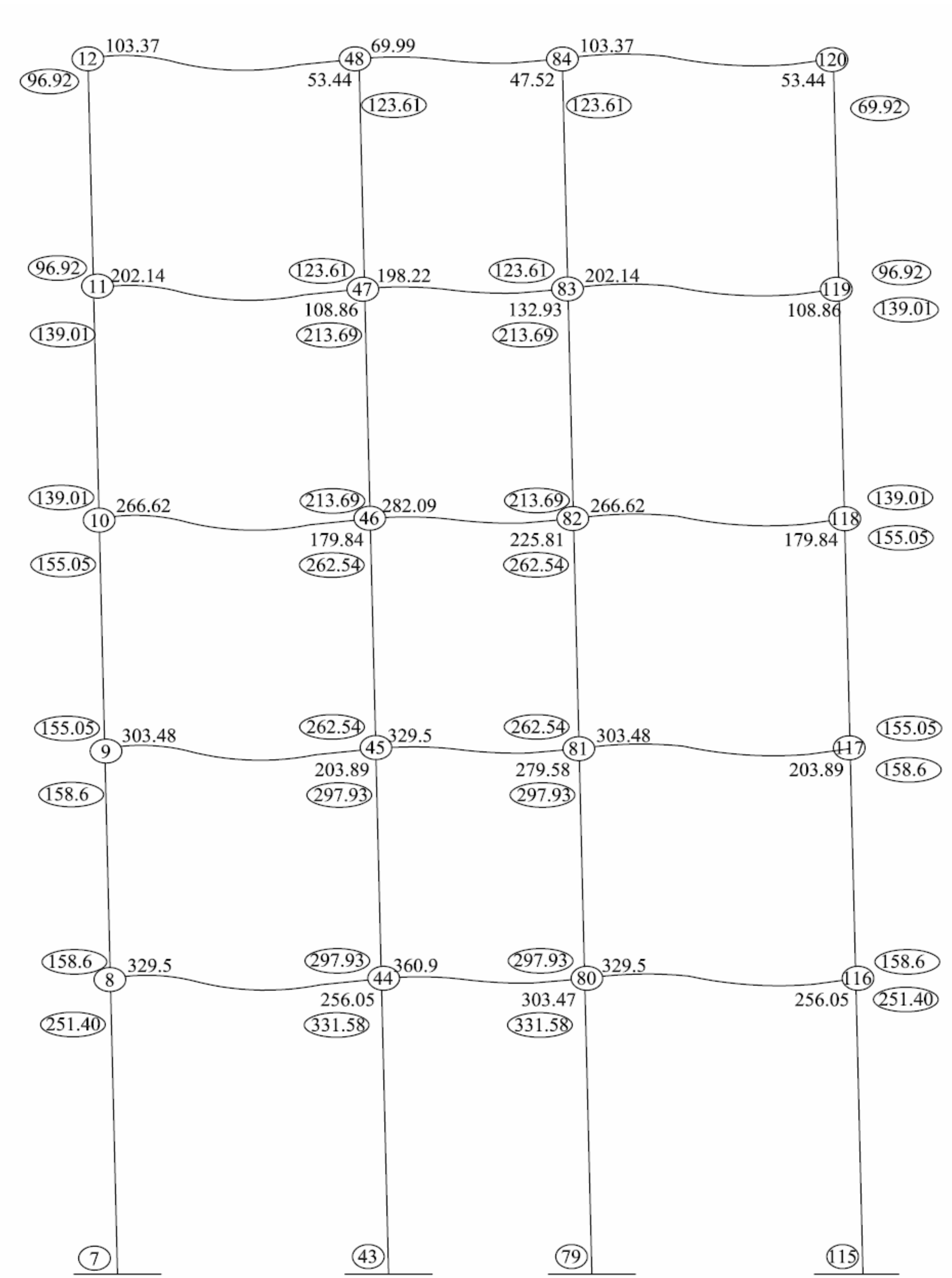
Axial Force & Biaxial Moments in Columns
Figure-27



Moment of Resistance of Beams as per Provided Reinforcement
Figure-28



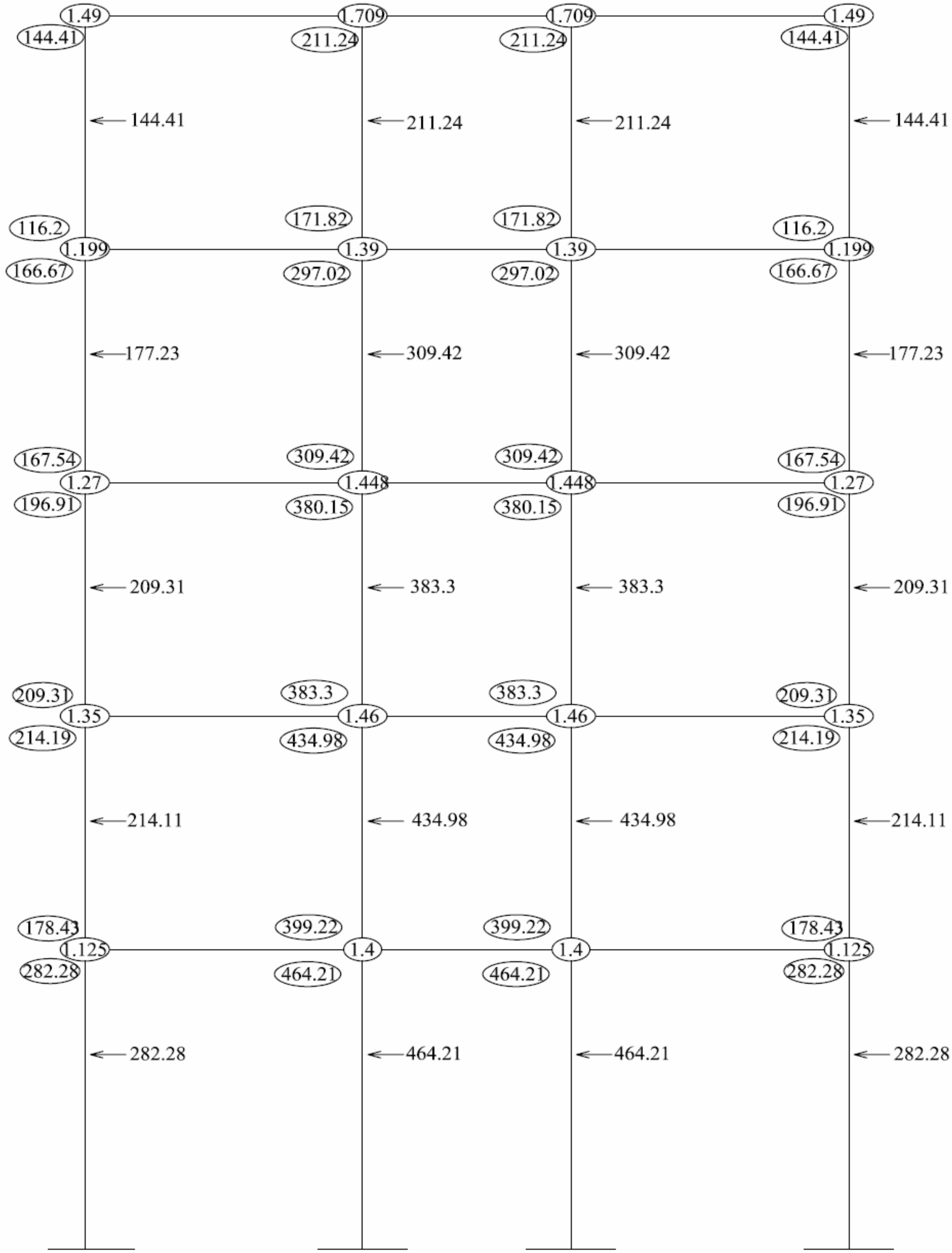
Seismic Direction 1
Figure-29



Seismic Direction 2
Figure-30

Table 6 Determination of moment magnification factor of columns of intermediate frame in XZ plane at all joints:

Joint No.	Seismic Direction	Sum of resisting moments of top & bottom of columns at joint. **1**	Sum of resisting moments of left & right beams at joint with an over strength factor 1.4. **2**	Check for **1≥2**	Moment Magnification factor **2÷1**
12	1	0+96.92 = 96.92	1.4(0+53.44) = 74.82	Ok	1.0
	2	0+96.92 = 96.92	1.4(0+103.37) = 144.71	Not Ok	1.49
48	1	0+123.61 = 123.61	1.4(103.37+47.52) = 211.25	Not Ok	1.709
	2	0+123.61 = 123.61	1.4(53.44+69.99) = 168.6	Not Ok	1.36
11	1	96.92+139.01 = 235.93	1.4(0+108.86) = 152.4	Ok	1.0
	2	96.92+139.01 = 235.93	1.4(0+202.14) = 282.996	Not Ok	1.199
47	1	123.61+213.69 = 337.3	1.4(202.14+132.93) = 469.09	Not Ok	1.39
	2	123.61+213.69 = 337.3	1.4(108.86+198.22) = 429.91	Not Ok	1.27
10	1	139.01+155.05 = 294.06	1.4(0+179.84) = 251.78	Ok	1.0
	2	139.01+155.05 = 294.06	1.4(0+266.62) = 373.27	Not Ok	1.27
46	1	213.69+262.54 = 476.23	1.4(266.62+225.81) = 689.4	Not Ok	1.448
	2	213.69+262.54 = 476.23	1.4(179.84+282.09) = 646.7	Not Ok	1.36
9	1	155.05+158.6 = 313.65	1.4(0+213.89) = 299.45	Ok	1.0
	2	155.05+158.6 = 313.65	1.4(0+303.48) = 424.87	Not Ok	1.35
45	1	262.54+297.93 = 560.47	1.4(303.48+282.09) = 819.9	Not Ok	1.46
	2	262.54+297.93 = 560.47	1.4(213.89+329.5) = 760.75	Not Ok	1.36
8	1	158.6+251.4 = 410	1.4(0+251.66) = 352.32	Ok	1.0
	2	158.6+251.4 = 410	1.4(0+329.5) = 461.3	Not Ok	1.125
44	1	297.93+331.58 = 629.51	1.4(329.5+303.47) = 886.16	Not Ok	1.4
	2	297.93+331.58 = 629.51	1.4(251.66+360.9) = 857.58	Not Ok	1.36



Revision of Column Moments According to Capacity Based Design by Moment Magnification Factor.
Figure-31

Table 7 Revised design capacity of columns of intermediate frame in XZ plane with earthquake force in X direction:

Storey No.	Column No	Column Size mm×mm	P_{uz} In KN	M_{ux} In KN	M_{uy} In KN	% of Steel	Interaction Ratio
5	10, 100	400×500	167.14	144.11	3.8	0.85	0.997
	40, 70	400×550	214.26	211.24	5.23	1.1	0.992
4	9, 99	400×500	438.55	177.23	9.98	1.0	0.984
	39, 69	400×550	564.89	309.63	13.8	1.7	0.994
3	8, 98	400×500	752.45	209.31	17.13	1.25	0.999
	38, 68	400×550	918.01	383.3	22.42	2.15	0.983
2	7, 97	400×500	1083.86	214.11	24.67	1.4	0.996
	37, 67	400×550	1274.06	434.98	31.12	2.55	0.996
1	6, 96	400×500	1323	282.28	32.77	2.3	0.995
	36, 66	400×550	1645.35	464.21	43.49	3.0	0.996

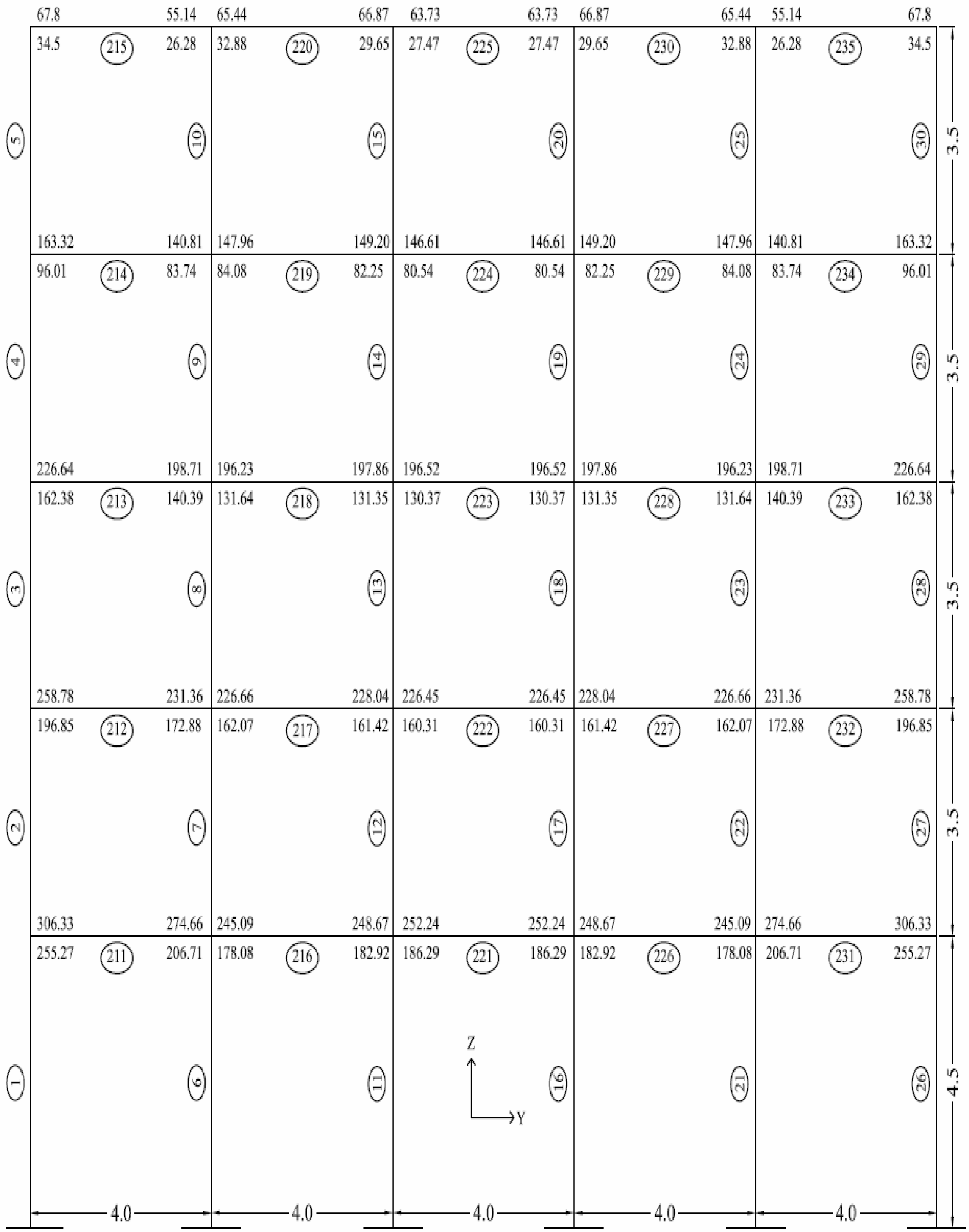
Table 8 Capacity based shear & shear reinforcement in beams of intermediate frame in XZ plane:

Beam No.	Shear in Seismic direction 1 In KN	Shear in Seismic direction 2 In KN	Maximum shear force In KN	Shear force from analysis In KN	Design shear force, V_u In KN	Shear Reinforcement Provided
140,150	-15.67 89.95	89.95 -15.67	89.95	74.78	89.95	8Φ two legged stirrup @ 95mm c/c.
145	-32.43 77.35	77.35 -32.43	77.35	60.12	77.35	8Φ two legged stirrup @ 95mm c/c.
139,149	-42.31 175.39	175.39 -42.31	175.39	148.49	175.39	8Φ two legged stirrup @ 95mm c/c.
144	-108.83 200.35	200.35 -108.83	200.35	155.18	200.35	8Φ two legged stirrup @ 95mm c/c.
138,148	-89.58 222.66	222.66 -89.58	222.66	179.89	222.66	8Φ two legged stirrup @ 100mm c/c.
143	-180.09 261.66	261.66 -180.09	261.66	210.67	261.66	8Φ two legged stirrup @ 85mm c/c.
137,147	-114.84 247.32	247.32 -114.84	247.32	196.67	247.32	8Φ two legged stirrup @ 90mm c/c.
142	-225.41 317.03	317.03 -2225.41	317.03	249.02	317.03	8Φ two legged stirrup @ 65mm c/c.
136,146	-136.87 269.95	269.95 -136.87	269.95	212.26	269.95	8Φ two legged stirrup @ 80mm c/c.
141	-247.23 336.85	336.85 -247.23	336.85	269.38	336.85	8Φ two legged stirrup @ 60mm c/c.

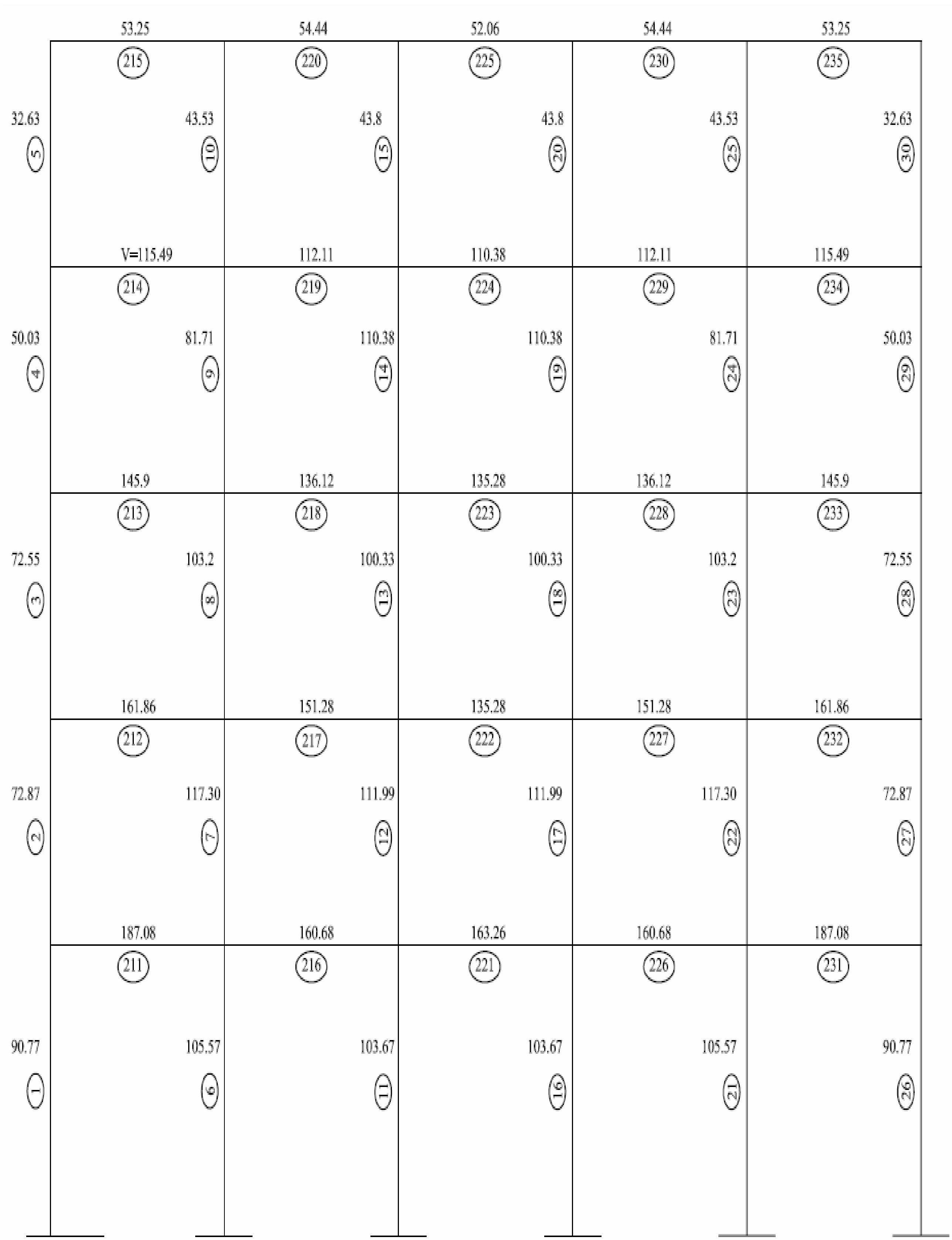
Chapter - 5.3

Design of End Frame in YZ Plane

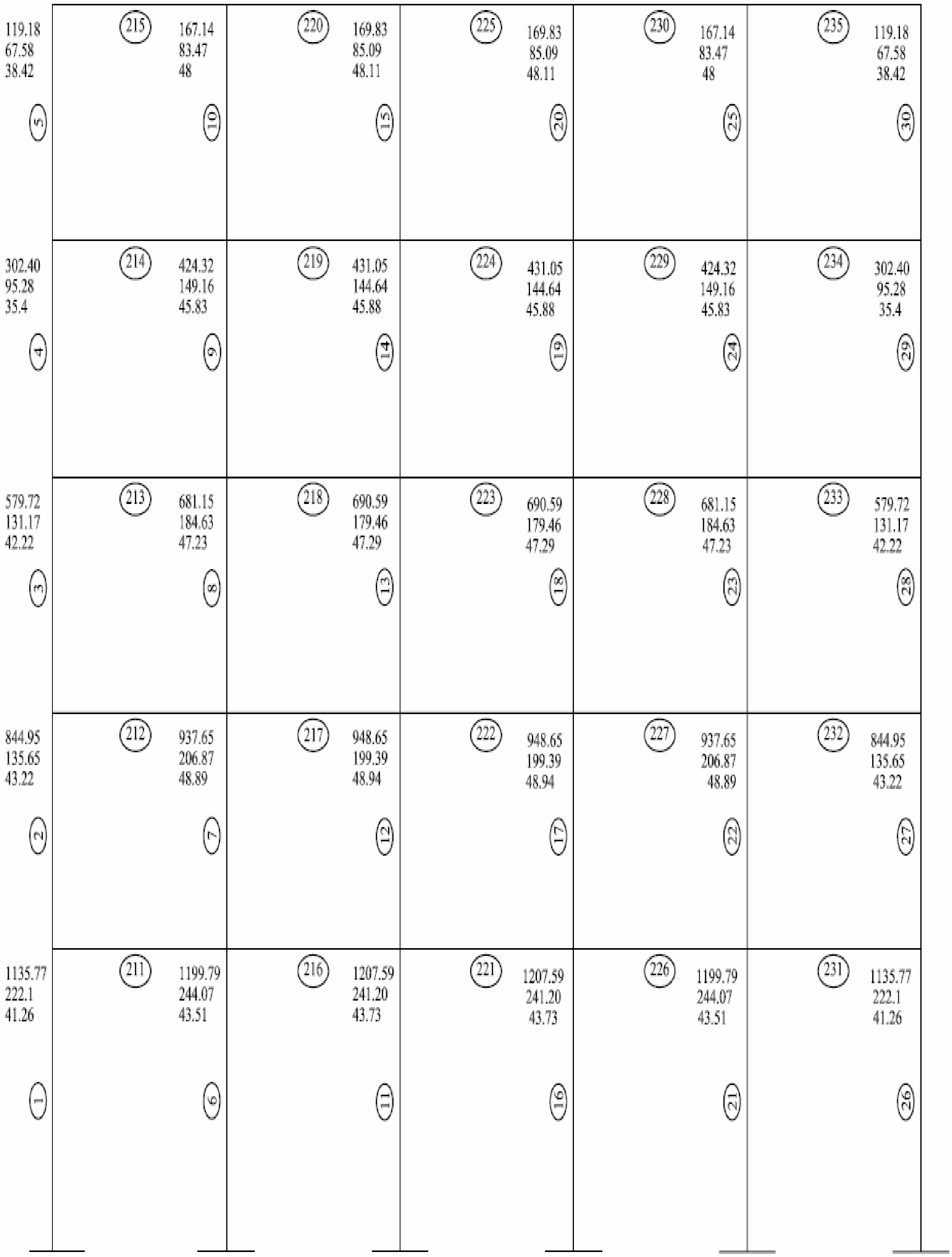
Design of End Frame in YZ Plane:



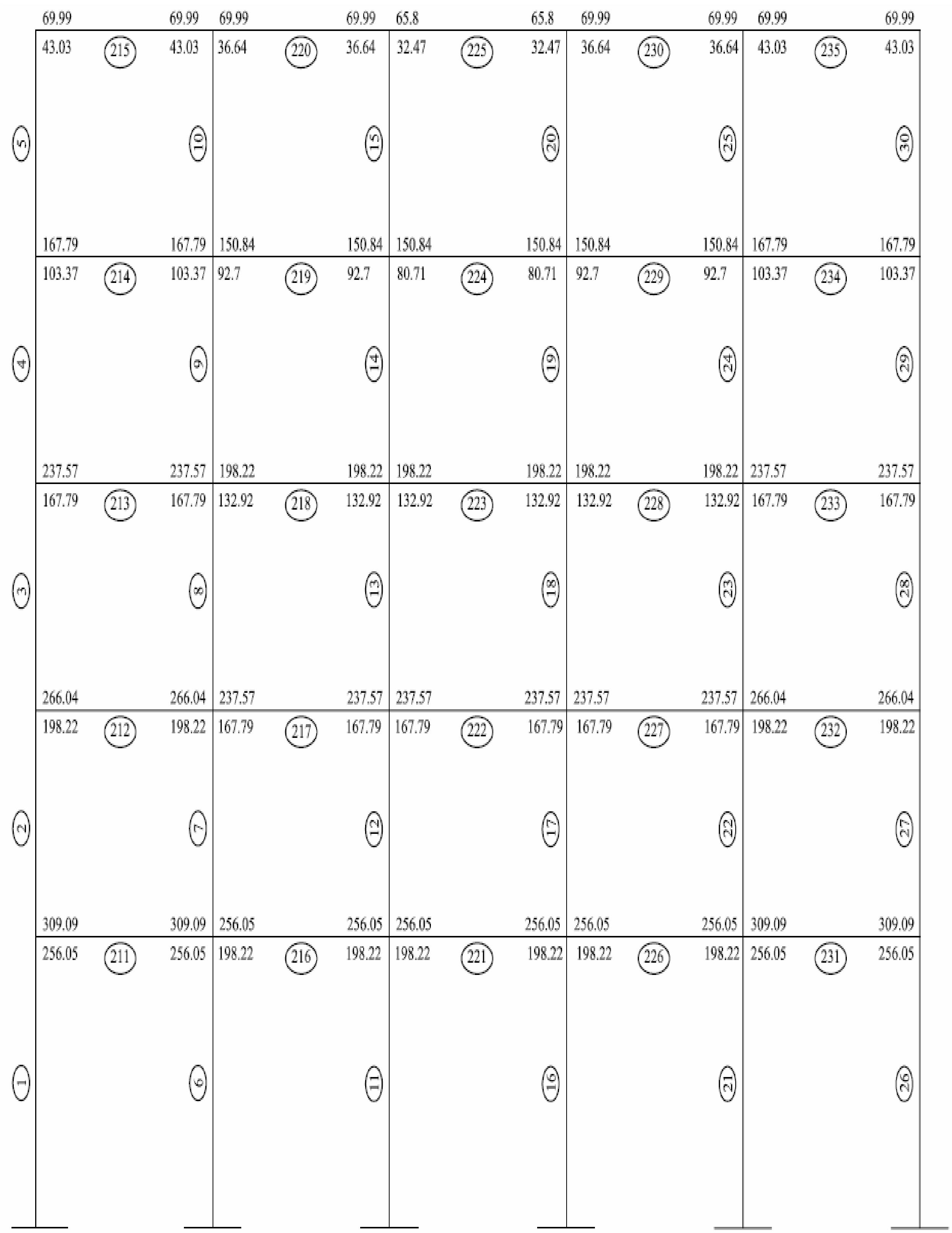
Maximum Sagging (Below) & Hogging (Above) Moments of Beams.
Figure-32



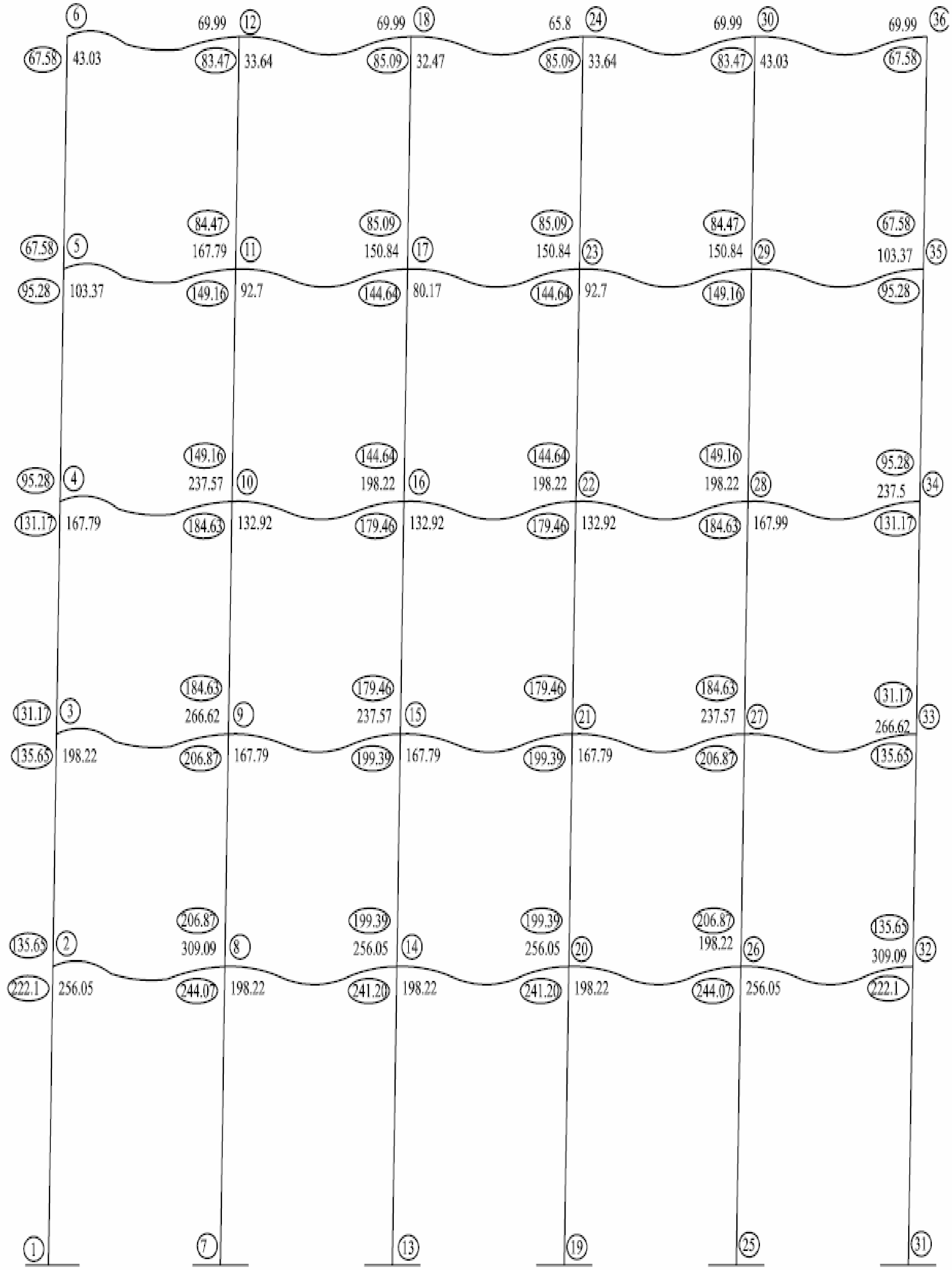
Maximum Shear Force
Figure-33



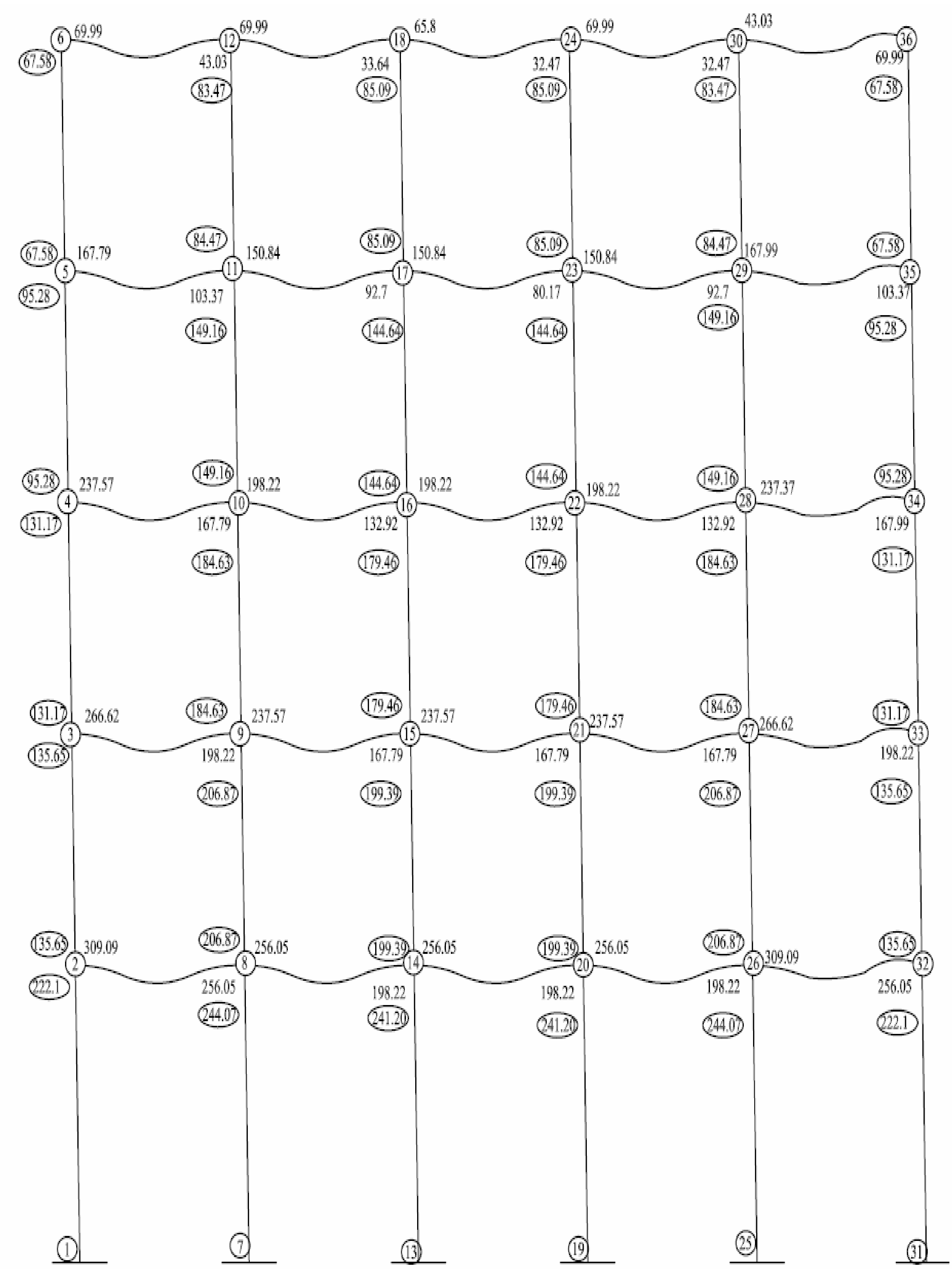
Axial Force & Biaxial Moments of Column
Figure-34



Moment of Resistance of Beams as Per Provided Reinforcement
Figure-35



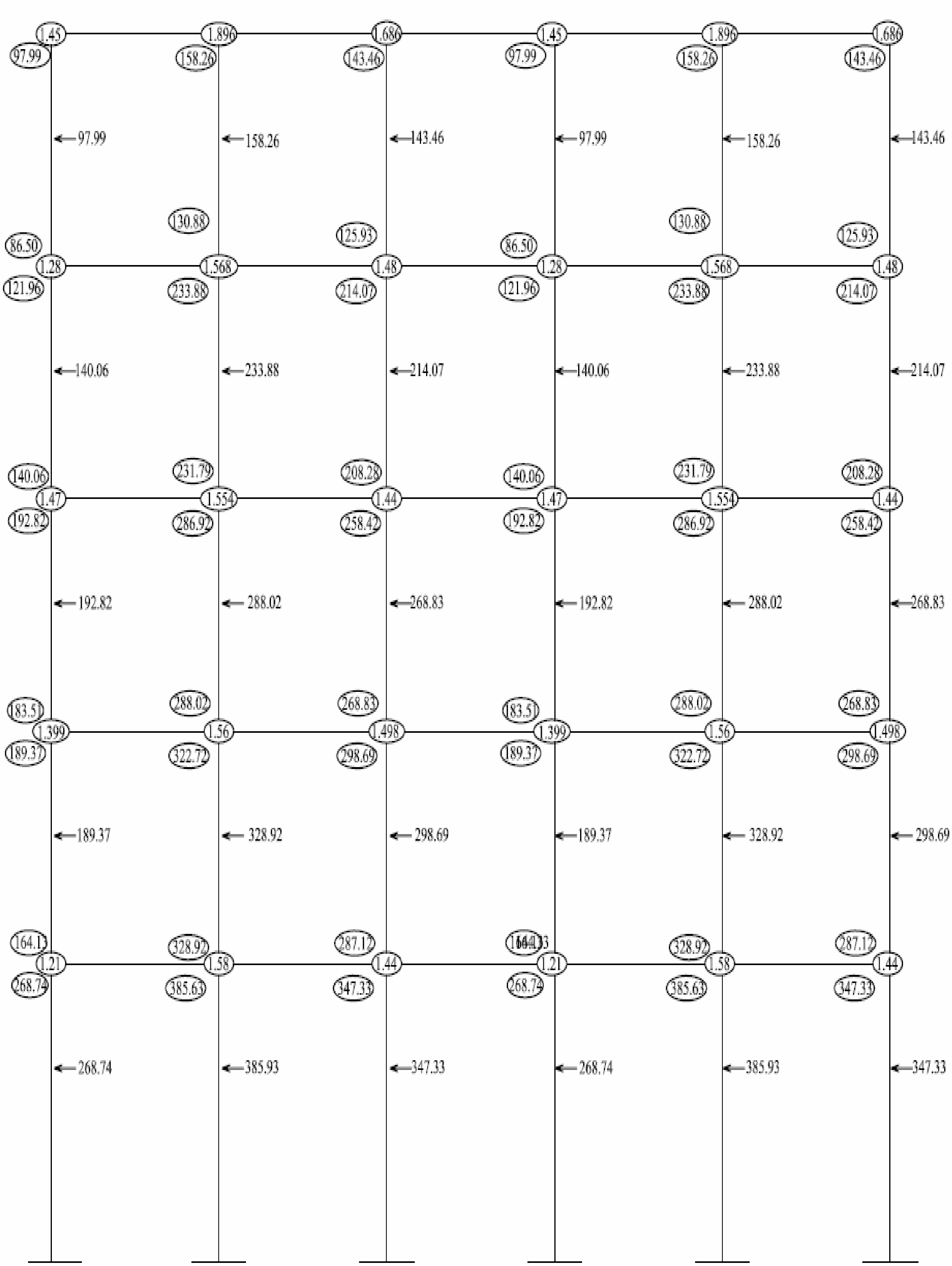
Seismic Direction 1
Figure-36



Seismic Direction 2
Figure-37

Table 9 Determination of moment magnification factor of columns of end frame in YZ plane at all joints:

Joint No.	Seismic Direction	Sum of resisting moments of top & bottom of columns at joint. **1**	Sum of resisting moments of left & right beams at joint with an over strength factor 1.4. **2**	Check for **1≥2**	Moment Magnification factor **2÷1**
6	1	0+67.58 = 67.58	1.4(0+43.03) = 60.24	Ok	1.0
	2	0+67.58 = 67.58	1.4(0+69.99) = 97.986	Not Ok	1.45
12	1	0+83.47 = 83.47	1.4(69.99+33.64) = 145.08	Not Ok	1.738
	2	0+83.47 = 83.47	1.4(43.03+69.99) = 158.23	Not Ok	1.896
18	1	0+85.09 = 85.08	1.4(69.99+32.47) = 143.44	Not Ok	1.686
	2	0+85.09 = 85.09	1.4(33.64+65.8) = 139.22	Not Ok	1.636
5	1	67.58+95.28 = 162.86	1.4(0+103.37) = 144.72	Ok	1.0
	2	67.58+95.28 = 162.86	1.4(0+149.16) = 208.82	Not Ok	1.28
11	1	83.47+149.16 = 232.63	1.4(167.79+92.7) = 364.69	Not Ok	1.568
	2	83.47+149.16 = 232.63	1.4(103.37+150.84) = 355.89	Not Ok	1.53
17	1	85.09+144.64 = 229.73	1.4(150.84+80.71) = 324.17	Not Ok	1.41
	2	85.09+144.64 = 229.73	1.4(92.7+150.84) = 340.96	Not Ok	1.48
4	1	95.28+131.17 = 226.45	1.4(0+167.79) = 234.91	Not Ok	1.04
	2	95.28+131.17 = 226.45	1.4(0+237.57) = 332.6	Not Ok	1.47
10	1	149.16+184.63 = 333.79	1.4(237.57+132.92) = 518.69	Not Ok	1.554
	2	149.16+184.63 = 333.79	1.4(167.79+198.22) = 512.42	Not Ok	1.535
16	1	144.64+177.46 = 322.1	1.4(198.22+132.92) = 463.6	Not Ok	1.44
	2	144.64+177.46 = 322.1	1.4(198.22+132.92) = 463.6	Not Ok	1.44
3	1	131.17+135.65 = 266.82	1.4(0+198.22) = 277.51	Not Ok	1.04
	2	131.17+135.65 = 266.82	1.4(0+266.62) = 373.27	Not Ok	1.399
9	1	184.63+206.87 = 391.5	1.4(266.62+167.79) = 608.17	Not Ok	1.55
	2	184.63+206.87 = 391.5	1.4(198.22+237.57) = 610.11	Not Ok	1.56
15	1	179.46+199.39 = 378.85	1.4(237.57+167.79) = 567.5	Not Ok	1.498
	2	179.46+199.39 = 378.85	1.4(167.79+237.57) = 567.5	Not Ok	1.498
2	1	135.65+222.1 = 357.75	1.4(0+256.05) = 358.47	Not Ok	1.002
	2	135.65+222.1 = 357.75	1.4(0+309.09) = 432.73	Not Ok	1.21
8	1	206.87+244.07 = 450.94	1.4(309.09+198.22) = 710.23	Not Ok	1.573
	2	206.87+244.07 = 450.94	1.4(256.05+256.05) = 716.94	Not Ok	1.58
14	1	199.39+241.2 = 440.59	1.4(256.05+198.22) = 635.98	Not Ok	1.44
	2	199.39+241.2 = 440.59	1.4(198.22+256.05) = 635.98	Not Ok	1.44



Revision of Column Moments According to Capacity Based Design by Moment Magnification Factor
Figure-38

Table 10 Revised design capacity of columns of end frame in YZ plane with earthquake force in Y direction:

Storey No.	Column No.	Column Size mm×mm	P_{uz} In KN	M_{ux} In KN	M_{uy} In KN	% of Steel	Interaction Ratio
5	5, 30	400×500	119.18	38.42	97.99	1.15	0.981
	10, 25	400×500	167.14	48	158.26	1.45	0.986
	15, 20	400×500	169.83	48.11	143.46	1.3	0.991
4	4, 29	400×500	302.4	35.4	140.06	1.4	0.993
	9, 24	400×500	424.32	45.83	233.88	1.675	0.997
	14, 19	400×500	431.05	45.88	224.07	1.525	0.995
3	3, 28	400×500	579.72	42.22	192.82	1.6	0.99
	8, 23	400×500	681.15	47.23	288.02	2.15	0.99
	13, 18	400×500	690.59	47.29	268.83	2.0	0.999
2	2, 27	400×500	844.95	43.22	190.59	1.825	0.988
	7, 22	400×500	937.65	48.89	322.72	2.6	0.992
	12, 17	400×500	948.65	48.94	298.69	2.35	0.992
1	1, 26	400×500	1135.77	41.26	268.74	2.65	0.99
	6, 21	400×500	1199.79	43.51	385.63	3.0	0.996
	11, 16	400×500	1207.59	43.73	347.33	2.8	0.99

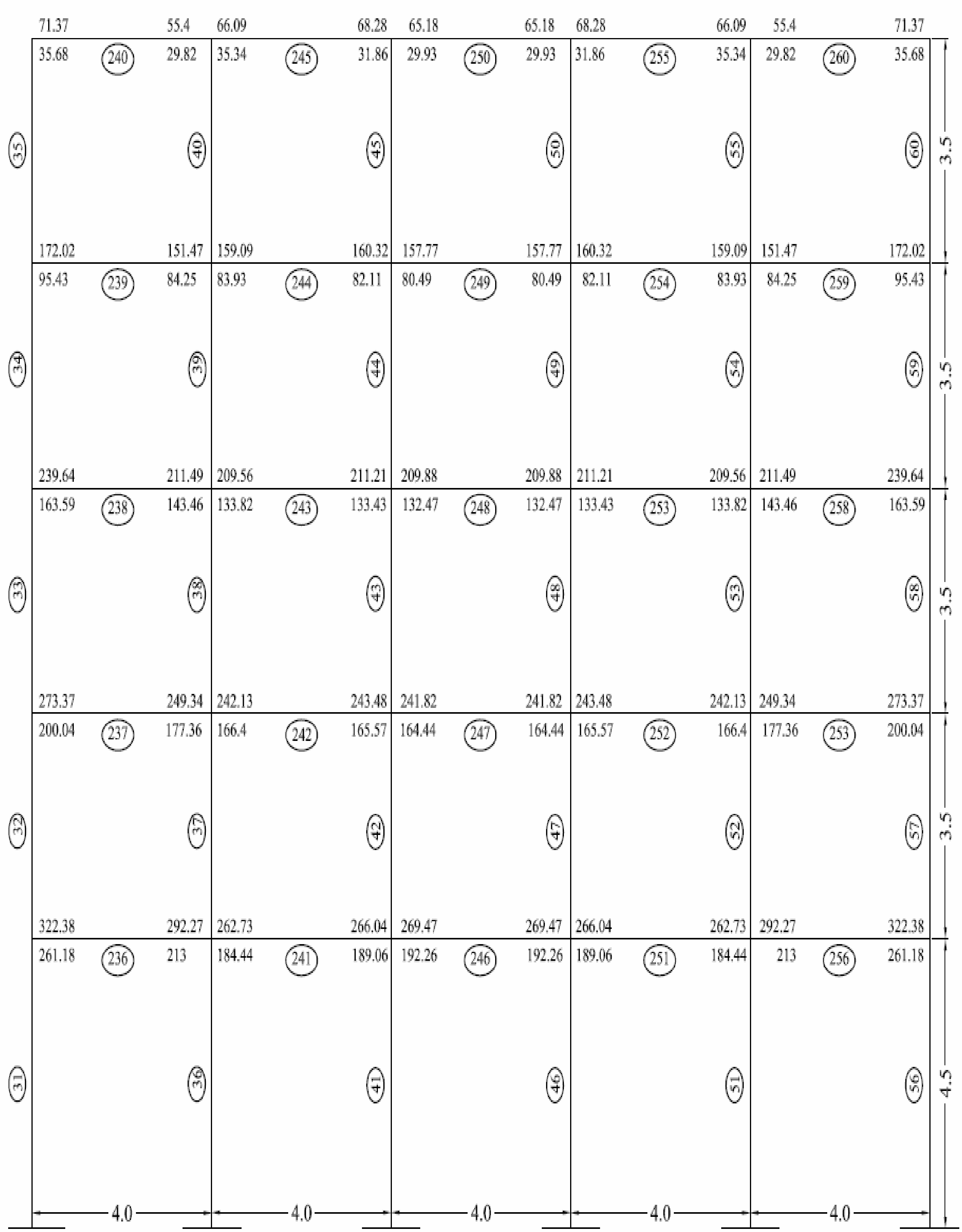
Table 11 Capacity based shear & shear reinforcement in beams of end frame in YZ plane:

Beam No.	Shear in Seismic direction 1 In KN	Shear in Seismic direction 2 In KN	Maximum shear force In KN	Shear force from analysis In KN	Design shear force, V_u In KN	Shear Reinforcement Provided
215,235	-16.93 62.18	62.18 -16.93	62.18	53.25	62.18	8Φ two legged stirrup @ 80mm c/c.
220,230	-13.65 58.89	58.89 -13.65	58.89	54.44	58.89	8Φ two legged stirrup @ 80mm c/c.
225	-11.77 57.01	57.01 -11.77	57.01	52.06	57.01	8Φ two legged stirrup @ 95mm c/c.
214,234	-39.29 150.53	150.53 -39.29	150.53	115.49	150.53	8Φ two legged stirrup @ 100mm c/c.
219,229	-29.62 140.86	140.86 -29.62	140.86	112.11	140.86	8Φ two legged stirrup @ 80mm c/c.
224	-25.42 136.66	136.66 25.42	136.66	110.38	136.66	8Φ two legged stirrup @ 100mm c/c.
213,233	-86.26 197.5	197.5 86.26	197.5	145.9	197.5	8Φ two legged stirrup @ 100mm c/c.
218,228	-60.28 171.52	171.52 -60.28	171.52	136.12	171.52	8Φ two legged stirrup @ 95mm c/c.
223	-60.28 171.52	171.52 -60.28	171.52	135.28	171.52	8Φ two legged stirrup @ 95mm c/c.
212,232	-107.07 218.31	218.31 -107.07	218.31	161.86	218.31	8Φ two legged stirrup @ 100mm c/c.
217,227	-86.26 197.5	197.5 -86.26	197.5	151.28	197.5	8Φ two legged stirrup @ 100mm c/c.
222	-86.26 197.5	197.5 -86.26	197.5	150.28	197.5	8Φ two legged stirrup @ 100mm c/c.
211,231	-142.18 253.42	253.42 -142.18	253.42	187.08	253.42	8Φ two legged stirrup @ 90mm c/c.
216,226	-103.37 214.61	214.61 -103.37	214.61	160.68	214.61	8Φ two legged stirrup @ 100mm c/c.
221	-103.37 214.61	214.61 -103.37	214.61	163.26	214.61	8Φ two legged stirrup @ 100mm c/c.

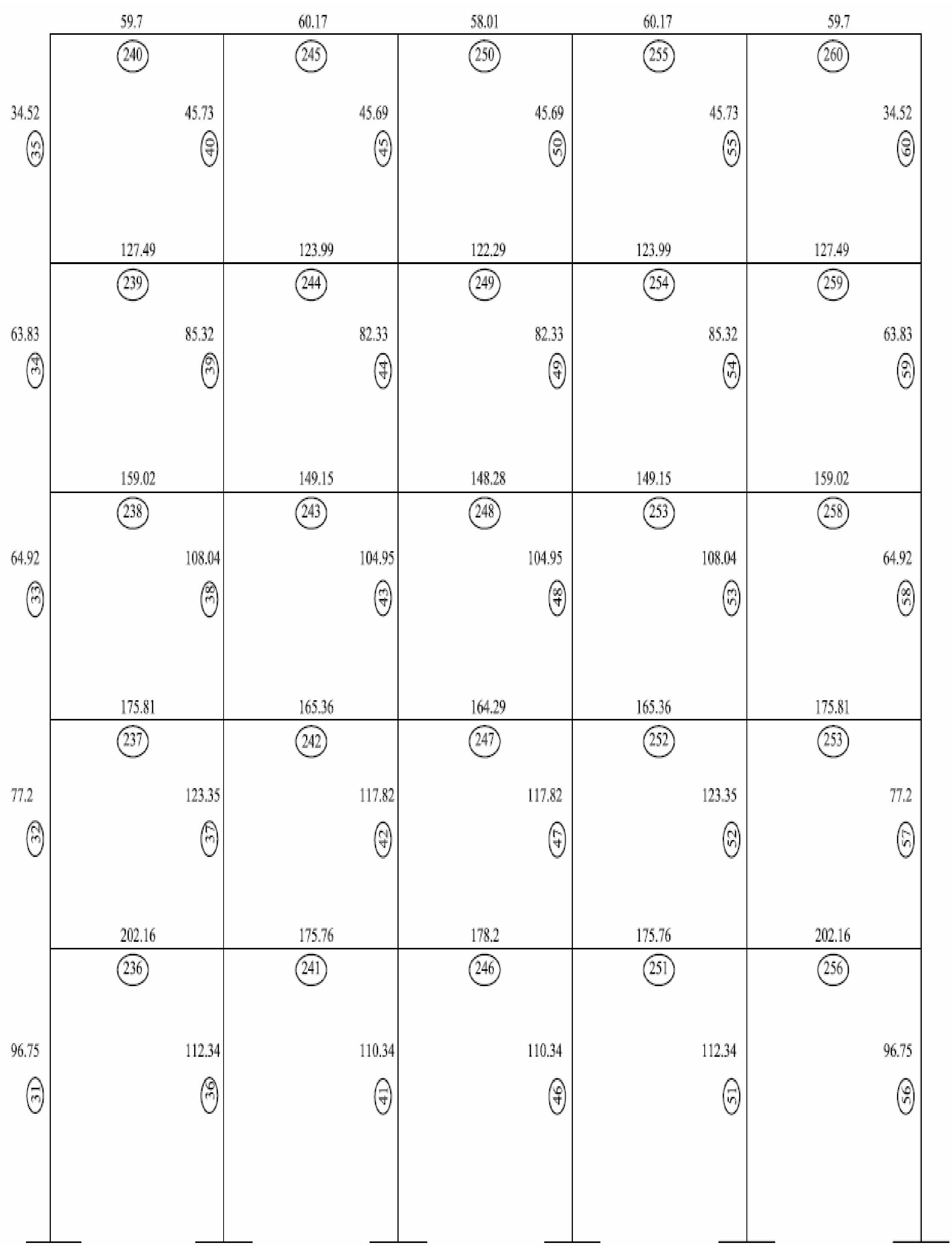
Chapter - 5.4

Design of Intermediate Frame in YZ Plane

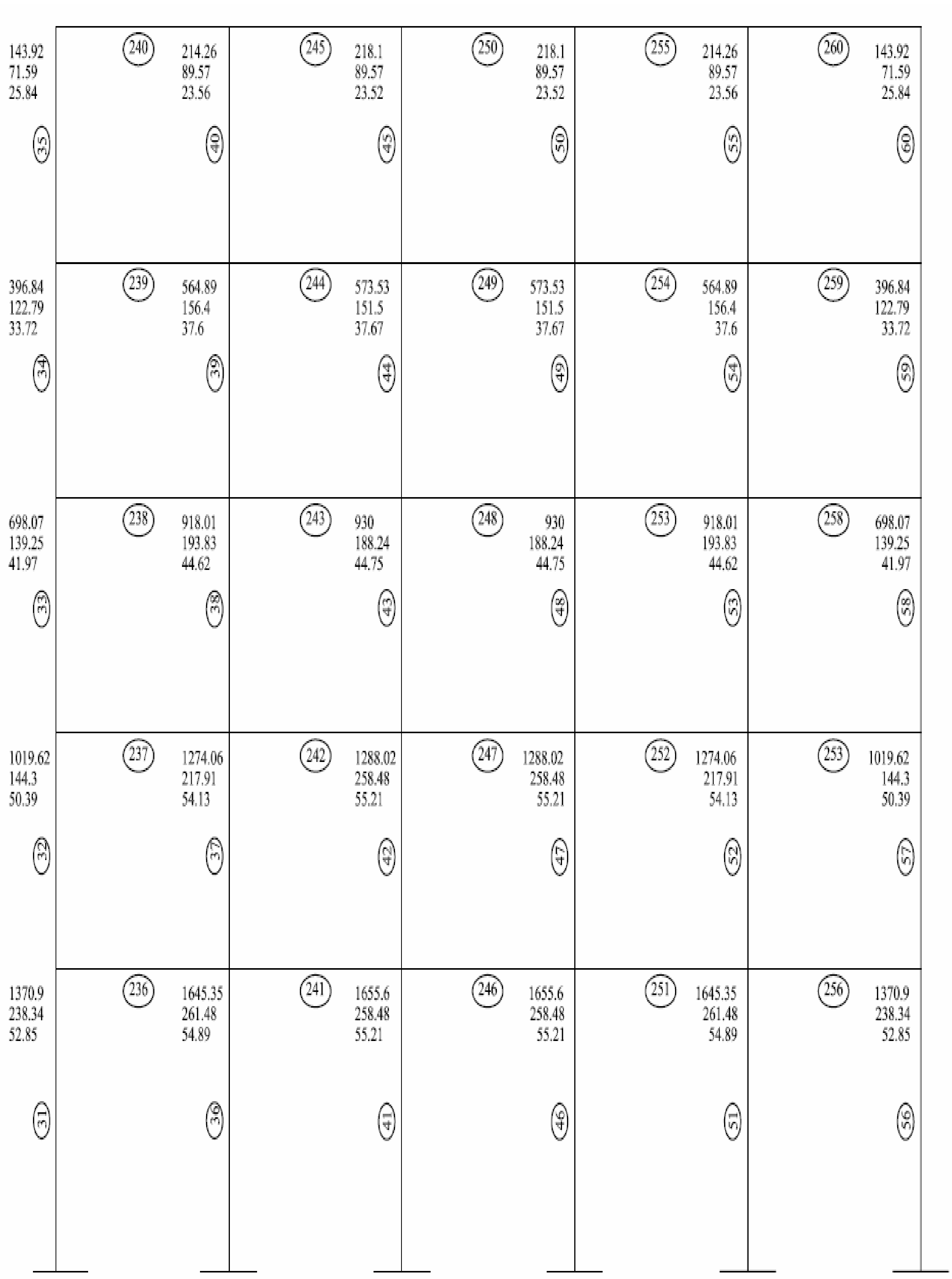
Design of Intermediate Frame in YZ Plane:



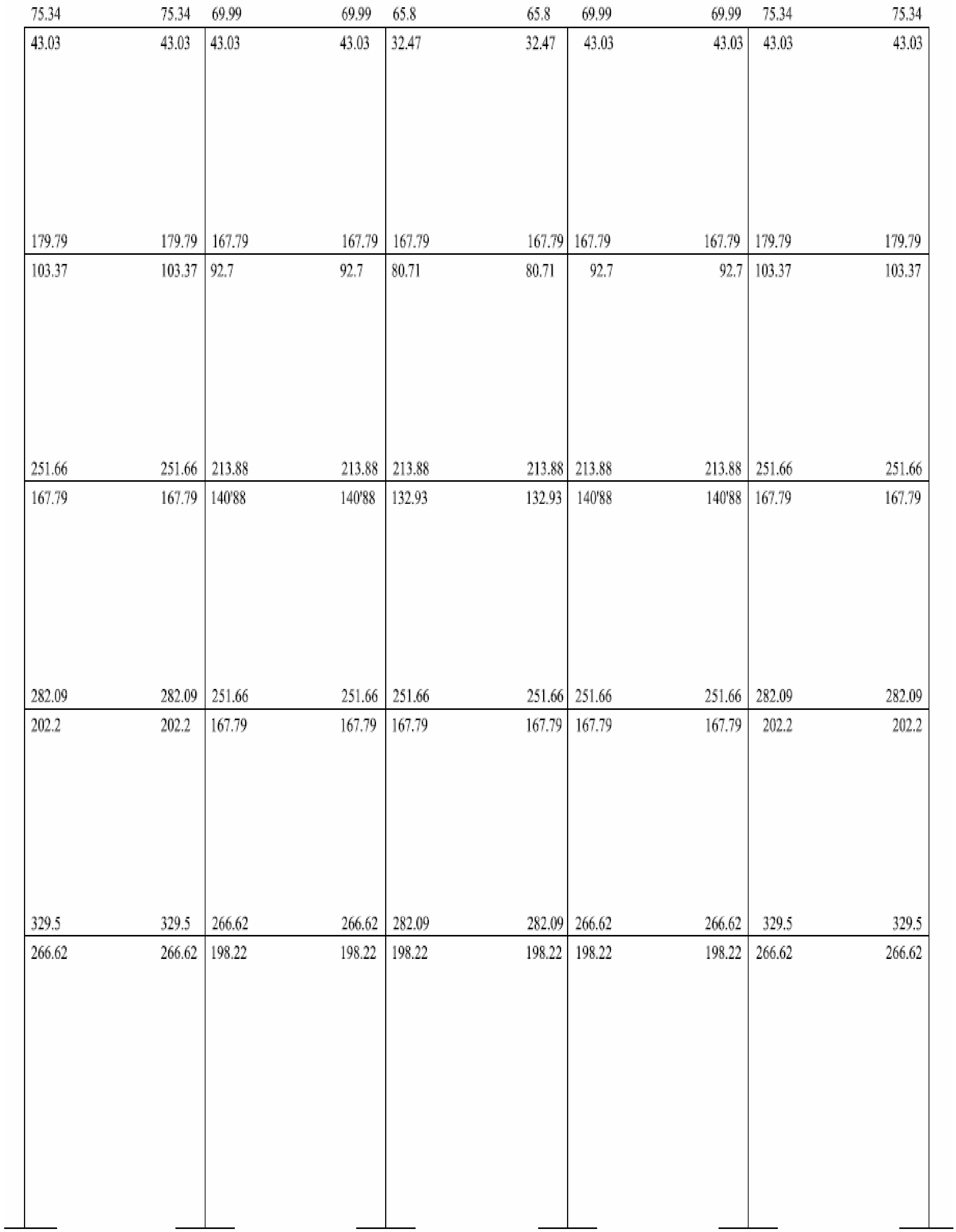
Maximum Sagging (Below) & Hogging (Above) Moments of Beams.
Figure-39



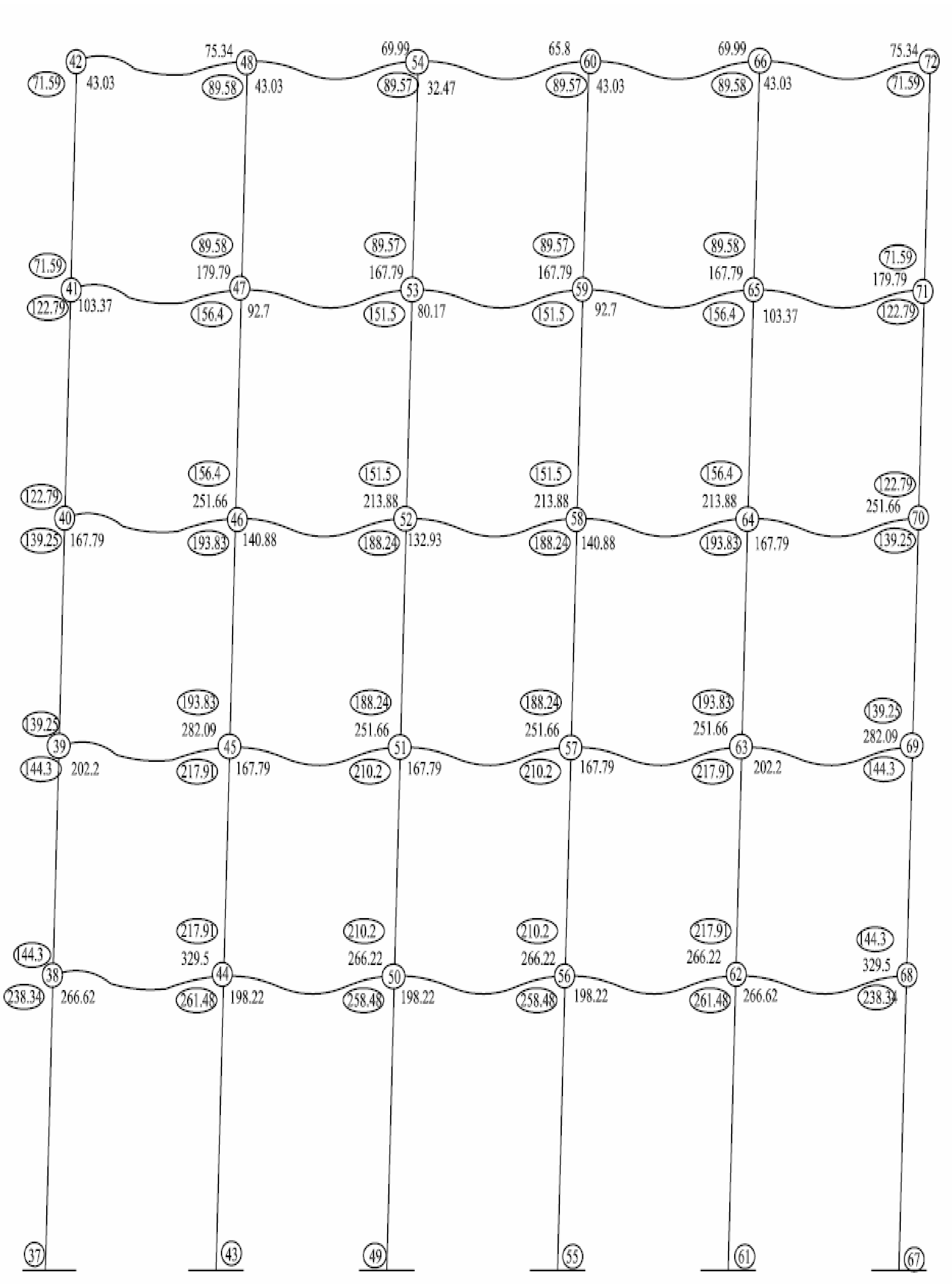
Maximum Shear Force
Figure-40



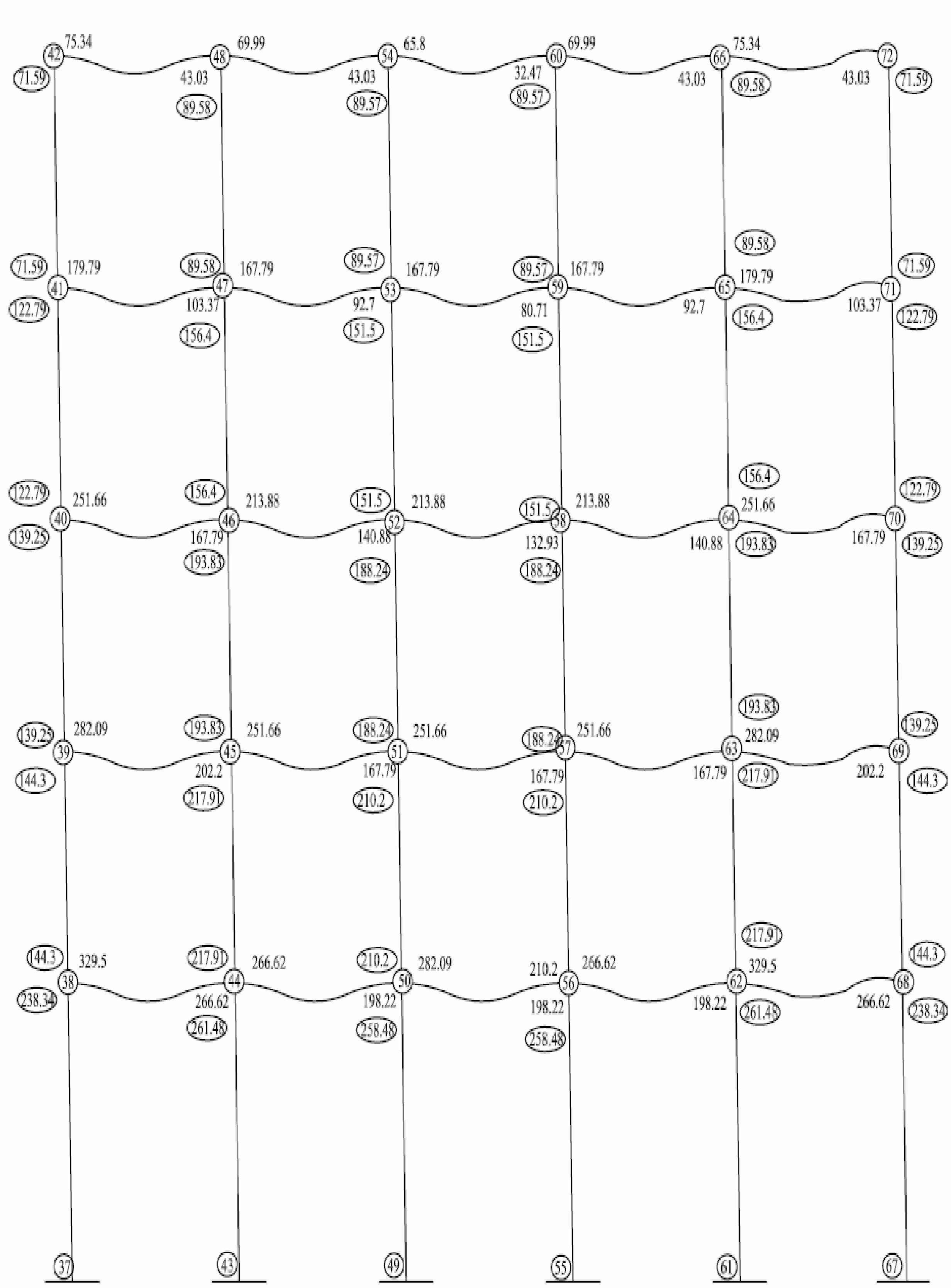
Axial Force & Biaxial Moments of Column
Figure-41



Moment of Resistance of Beams as Per Provided Reinforcement
Figure-42



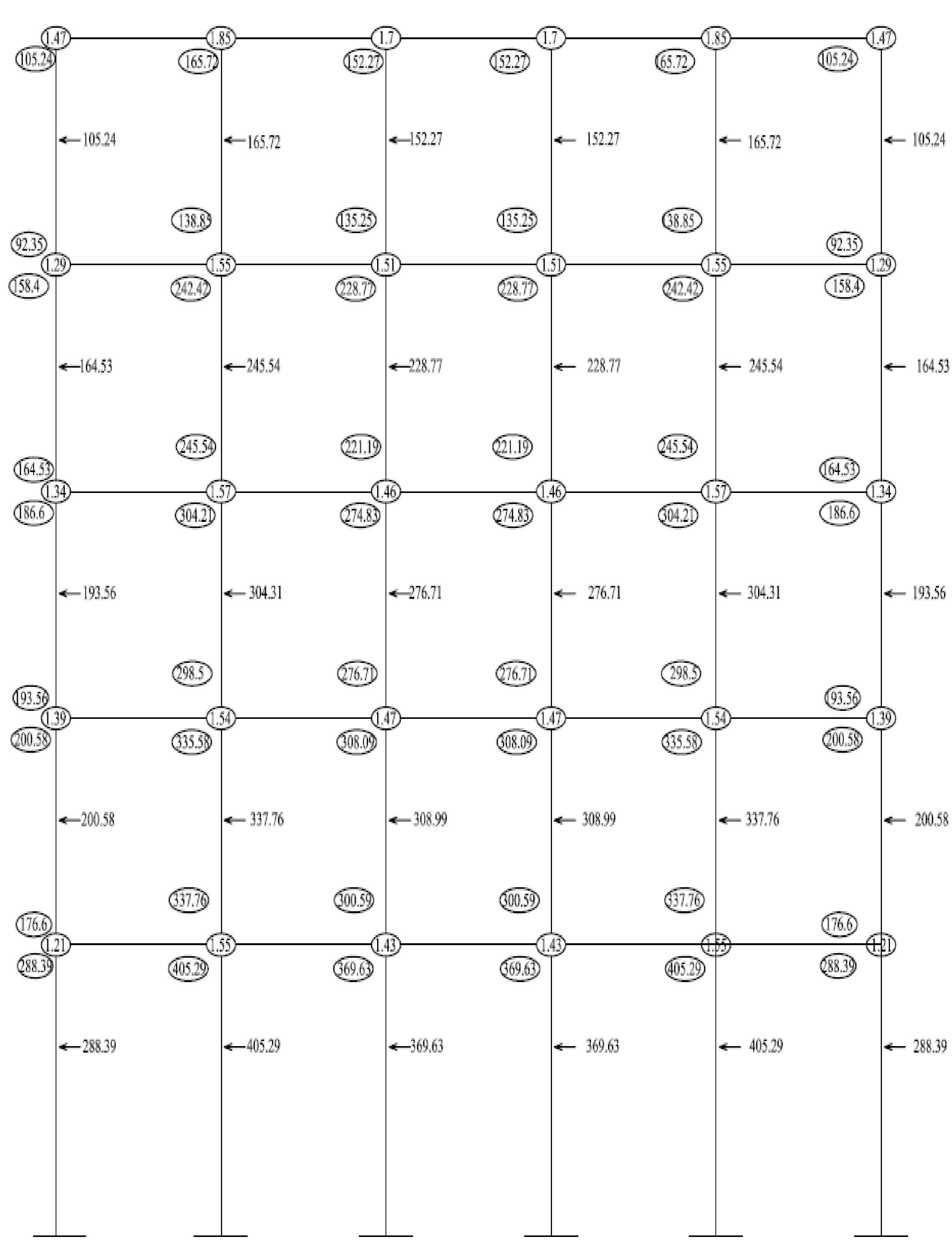
Seismic Direction 1
Figure-43



Seismic Direction 2
Figure-44

Table 12 Determination of moment magnification factor of columns of intermediate frame in YZ plane at all joints:

Joint No.	Seismic Direction	Sum of resisting moments of top & bottom of columns at joint. **1**	Sum of resisting moments of left & right beams at joint with an over strength factor 1.4. **2**	Check for **1≥2**	Moment Magnification factor **2÷1**
6	1	0+67.58 = 67.58	1.4(0+43.03) = 60.24	Ok	1.0
	2	0+67.58 = 67.58	1.4(0+69.99) = 97.986	Not Ok	1.45
12	1	0+83.47 = 83.47	1.4(69.99+33.64) = 145.08	Not Ok	1.738
	2	0+83.47 = 83.47	1.4(43.03+69.99) = 158.23	Not Ok	1.896
18	1	0+85.09 = 85.08	1.4(69.99+32.47) = 143.44	Not Ok	1.686
	2	0+85.09 = 85.09	1.4(33.64+65.8) = 139.22	Not Ok	1.636
5	1	67.58+95.28 = 162.86	1.4(0+103.37) = 144.72	Ok	1.0
	2	67.58+95.28 = 162.86	1.4(0+149.16) = 208.82	Not Ok	1.28
11	1	83.47+149.16 = 232.63	1.4(167.79+92.7) = 364.69	Not Ok	1.568
	2	83.47+149.16 = 232.63	1.4(103.37+150.84) = 355.89	Not Ok	1.53
17	1	85.09+144.64 = 229.73	1.4(150.84+80.71) = 324.17	Not Ok	1.41
	2	85.09+144.64 = 229.73	1.4(92.7+150.84) = 340.96	Not Ok	1.48
4	1	95.28+131.17 = 226.45	1.4(0+167.79) = 234.91	Not Ok	1.04
	2	95.28+131.17 = 226.45	1.4(0+237.57) = 332.6	Not Ok	1.47
10	1	149.16+184.63 = 333.79	1.4(237.57+132.92) = 518.69	Not Ok	1.554
	2	149.16+184.63 = 333.79	1.4(167.79+198.22) = 512.42	Not Ok	1.535
16	1	144.64+177.46 = 322.1	1.4(198.22+132.92) = 463.6	Not Ok	1.44
	2	144.64+177.46 = 322.1	1.4(198.22+132.92) = 463.6	Not Ok	1.44
3	1	131.17+135.65 = 266.82	1.4(0+198.22) = 277.51	Not Ok	1.04
	2	131.17+135.65 = 266.82	1.4(0+266.62) = 373.27	Not Ok	1.399
9	1	184.63+206.87 = 391.5	1.4(266.62+167.79) = 608.17	Not Ok	1.55
	2	184.63+206.87 = 391.5	1.4(198.22+237.57) = 610.11	Not Ok	1.56
15	1	179.46+199.39 = 378.85	1.4(237.57+167.79) = 567.5	Not Ok	1.498
	2	179.46+199.39 = 378.85	1.4(167.79+237.57) = 567.5	Not Ok	1.498
2	1	135.65+222.1 = 357.75	1.4(0+256.05) = 358.47	Not Ok	1.002
	2	135.65+222.1 = 357.75	1.4(0+309.09) = 432.73	Not Ok	1.21
8	1	206.87+244.07 = 450.94	1.4(309.09+198.22) = 710.23	Not Ok	1.573
	2	206.87+244.07 = 450.94	1.4(256.05+256.05) = 716.94	Not Ok	1.58
14	1	199.39+241.2 = 440.59	1.4(256.05+198.22) = 635.98	Not Ok	1.44
	2	199.39+241.2 = 440.59	1.4(198.22+256.05) = 635.98	Not Ok	1.44



Revision of Column Moments According to Capacity Based Design by Moment Magnification Factor
Figure-45

Table 13 Revised design capacity of columns of intermediate frame in YZ plane with earthquake force in Y direction:

Storey No.	Column No.	Column Size mm×mm	P_{uz} In KN	M_{ux} In KN	M_{uy} In KN	% of Steel	% of Steel
5	35, 60	400×550	143.92	25.84	105.24	1.375	0.701
	40, 55	400×550	214.26	23.56	165.72	1.6	0.998
	45, 50	400×550	218.1	23.52	152.27	1.425	0.992
4	34, 59	400×550	396.84	33.72	164.53	1.8	0.827
	39, 54	400×550	564.89	37.6	245.54	2.0	0.996
	44, 49	400×550	573.53	37.67	228.77	1.9	0.992
3	33, 58	400×550	698.07	41.97	193.56	2.2	0.978
	38, 53	400×550	918.01	44.62	304.31	2.4	0.996
	43, 48	400×550	930	44.75	276.71	2.275	0.996
2	32, 57	400×550	1019.62	50.39	200.58	2.5	0.707
	37, 52	400×550	1274.06	54.13	337.76	2.75	0.991
	42, 47	400×550	1288.02	54.43	308.99	2.65	0.984
1	31, 56	400×550	1370.9	52.85	288.39	3.0	0.997
	36, 51	400×550	1645.35	54.89	405.29	3.375	0.999
	41,46	400×550	1655.6	55.21	369.63	3.2	0.996

Table 14 Capacity based shear & shear reinforcement in beams of intermediate frame in YZ plane:

Beam No.	Shear in Seismic direction 1 In KN	Shear in Seismic direction 2 In KN	Maximum shear force In KN	Shear force from analysis In KN	Design shear force, V_u In KN	Shear Reinforcement Provided
240,260	-5.19 77.67	77.67 -5.19	77.67	59.7	77.67	8Φ two legged stirrup @ 80mm c/c.
245,255	-3.32 75.78	75.78 -3.32	75.78	60.17	75.78	8Φ two legged stirrup @ 80mm c/c.
250	1.828 70.65	70.65 1.828	70.65	58.01	70.65	8Φ two legged stirrup @ 95mm c/c.
239,259	-33.25 164.96	164.96 -33.25	164.96	127.49	164.96	8Φ two legged stirrup @ 100mm c/c.
244,254	-25.32 157.03	157.03 -25.32	157.03	123.99	157.03	8Φ two legged stirrup @ 80mm c/c.
249	-21.12 152.83	152.83 -21.12	152.83	122.29	152.83	8Φ two legged stirrup @ 100mm c/c.
238,258	-80.95 212.66	212.66 -80.95	212.66	159.02	212.66	8Φ two legged stirrup @ 100mm c/c.
243,253	-58.31 190.02	190.02 -58.31	190.02	149.15	190.02	8Φ two legged stirrup @ 100mm c/c.
248	-55.53 187.24	187.24 -55.53	187.24	148.28	187.24	8Φ two legged stirrup @ 95mm c/c.
237,257	-103.65 235.36	236.36 -103.65	235.36	175.81	235.36	8Φ two legged stirrup @ 95mm c/c.
242,252	-80.95 212.66	212.66 -80.95	212.66	165.36	212.66	8Φ two legged stirrup @ 100mm c/c.
247	-80.95 212.66	212.66 -80.95	212.66	165.36	212.66	8Φ two legged stirrup @ 100mm c/c.
236,256	-142.9 274.5	274.5 -142.9	274.5	202.16	274.5	8Φ two legged stirrup @ 80mm c/c.
241,251	-96.85 228.55	228.55 -96.85	228.55	175.76	228.55	8Φ two legged stirrup @ 100mm c/c.
246	-102.25 233.96	233.96 -102.25	233.96	178.2	233.96	8Φ two legged stirrup @ 95mm c/c.

Chapter -6

Final Design of Columns

Final Design of Column:

Table 15 Final design of columns of end frame in XZ plane:

Storey No.	Column No.	Column Size mm×mm	% of Steel	Interaction Ratio
5	5, 95 35, 65	400×500 400×550	1.15 1.375	0.985 0.998
4	4, 94 34, 64	400×500 400×550	1.4 1.8	0.993 0.995
3	3, 93 33,63	400×500 400×550	1.6 2.2	0.99 0.996
2	2, 92 32, 62	400×500 400×550	1.825 2.5	0.988 0.991
1	1, 91 31, 61	400×500 400×550	2.65 3.0	0.99 0.997

Table 16 Reinforcement provided in the columns of end frame in XZ plane:

Column No.	A_{st} Required	Bars Provided	A_{st} Provided
5, 95	2300	10×16Φ + 4×12Φ	2462.96
35, 65	3025	4×20Φ + 8×16Φ + 2×12Φ	3091.26
4, 94	2800	4×20Φ + 6×16Φ + 4×12Φ	2915.32
34, 64	3960	4×25Φ + 4×20Φ + 2×16Φ + 4×12Φ	4074.56
3, 93	3200	4×20Φ + 10×16Φ	3267.2
33, 63	4840	8×25Φ + 4×16Φ + 2×12Φ	4957.38
2, 92	3650	8×20Φ + 6×16Φ	3719.56
32, 62	5500	8×25Φ + 4×20Φ + 2×16Φ	5585.68
1, 91	5300	6×25Φ + 8×20Φ	5458.42
31, 61	6600	4×30Φ + 4×25Φ + 6×20Φ	6675.78

Table 17 Capacity based shear in columns of end frame in XZ plane:

Column No.	Capacity based shear in KN.
5, 95	1.4[(135.06 + 135.06)/ 3.5] = 108.04
35, 65	1.4[(213.34 + 213.34)/ 3.5] = 170.67
4, 94	1.4[(168.43+ 168.43)/ 3.5] = 134.74
34, 64	1.4[(300.75 + 300.75)/ 3.5] = 240.6
3, 93	1.4[(210.21 + 210.21)/ 3.5] = 168.17
33, 63	1.4[(380.17 + 380.17)/ 3.5] = 304.14
2, 92	1.4[(214.77 + 214.77)/ 3.5] = 171.82
32, 62	1.4[(430.125 + 430.125)/ 3.5] = 344.1
1, 91	1.4[(285.13 + 285.13)/ 4.5] = 177.41
31,61	1.4[(468.22 + 468.22)/ 4.5] = 291.3

Table 18 Shear reinforcement and special confining reinforcement for columns of end frame in XZ plane:

Column No.	L_0 In mm	Shear reinforcement provided.	Special confining reinforcement provided.
5, 95	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
35, 65	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
4, 94	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
34, 64	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
3, 93	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
33, 63	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
2, 92	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
32, 62	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
1, 91	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
31, 61	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.

Table 19 Final design of columns of intermediate frame next to end frame in XZ plane:

Storey No.	Column No.	Column Size mm×mm	% of Steel	Interaction Ratio
5	10, 100 40, 70	400×500 400×550	1.45 1.6	0.986 0.998
4	9, 99 39, 69	400×500 400×550	1.675 2.0	0.997 0.996
3	8, 98 38, 68	400×500 400×550	2.15 2.4	0.99 0.996
2	7, 97 37, 67	400×500 400×550	2.6 2.75	0.992 0.996
1	6, 96 36, 66	400×500 400×550	3.0 3.375	0.999 0.999

Table 20 Reinforcement provided in the columns of intermediate frame next to end frame in XZ plane:

Column No.	A_{st} Required	Bars Provided	A_{st} Provided
10, 100	2900	4×20Φ + 6×16Φ + 4×12Φ	2916.52
40, 70	3520	8×20Φ + 4×16Φ + 2×12Φ	3546.42
9, 99	3350	8×20Φ + 2×16Φ + 4×12Φ	3370.08
39, 69	4400	4×25Φ + 4×20Φ + 6×16Φ	4424.44
8, 98	4300	4×25Φ + 6×20Φ + 4×12Φ	4300.74
38, 68	5280	8×25Φ + 2×20Φ + 4×16Φ	5359.5
7, 97	5200	8×25Φ + 2×20Φ + 4×16Φ	5359.5
37, 72	6050	10×25Φ + 4×20Φ	6165.3
6, 96	6000	10×25Φ + 4×20Φ	6165.3
36, 66	7425	6×30Φ + 4×25Φ + 4×20Φ	7461.18

Table 21 Capacity based shear in columns of frame next to end frame in XZ plane:

Column No.	Capacity based shear in KN.
10, 100	1.4[(158.26 + 158.26)/ 3.5] = 126.61
40, 70	1.4[(211.24 + 211.24)/ 3.5] = 168.99
9, 99	1.4[(233.88+ 233.88)/ 3.5] = 187.1
39, 69	1.4[(309.63 + 309.63)/ 3.5] = 247.7
8, 98	1.4[(288.02 + 288.02)/ 3.5] = 230.42
38, 68	1.4[(380.68 + 380.68)/ 3.5] = 304.54
7, 97	1.4[(322.72 + 322.72)/ 3.5] = 258.18
37, 67	1.4[(431.99 + 431.99)/ 3.5] = 345.59
6, 96	1.4[(385.67 + 385.67)/ 4.5] = 239.97
36,66	1.4[(464.51 + 464.51)/ 4.5] = 289.03

Table 22 Shear reinforcement and special confining reinforcement for columns of frame next to end frame in XZ plane:

Column No.	L_0 In mm	Shear reinforcement provided.	Special confining reinforcement provided.
10, 100	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
40, 70	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
9, 99	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
39, 69	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
8, 98	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
38, 68	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
7, 97	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
37, 67	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
6, 96	675	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
36,66	675	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.

Table 23 Final design of columns of intermediate frame in XZ plane:

Storey No.	Column No.	Column Size mm×mm	% of Steel	Interaction Ratio
5	15, 105 45, 75	400×500 400×550	1.3 1.425	0.991 0.992
4	14, 104 44, 74	400×500 400×550	1.525 1.9	0.995 0.998
3	13, 103 43,73	400×500 400×550	2.0 2.275	0.999 0.996
2	12, 102 42, 72	400×500 400×550	2.35 2.6	0.992 0.985
1	11, 101 41, 71	400×500 400×550	2.8 3.2	0.992 0.996

Table 24 Reinforcement provided in the columns of end frame in XZ plane:

Column No.	A_{st} Required	Bars Provided	A_{st} Provided
15, 105	2600	4×20Φ + 4×16Φ + 6×12Φ	2739.38
45, 75	3235	4×20Φ + 4×16Φ + 4×12Φ	3243.3
14, 104	3050	4×20Φ + 8×16Φ + 2×12Φ	3091.26
44, 74	4180	6×25Φ + 4×16Φ + 4×12Φ	4201.81
13, 103	4000	4×25Φ + 4×20Φ + 2×16Φ + 4×12Φ	4074.56
43, 73	5005	6×25Φ + 4×20Φ + 4×16Φ	5006.06
12, 102	4700	4×25Φ + 8×20Φ + 2×12Φ	4702.86
42, 72	5720	8×25Φ + 6×20Φ	5811.86
11, 101	5600	8×25Φ + 6×20Φ	5811.86
41, 71	7040	4×30Φ + 8×25Φ + 2×16Φ	7156.48

Table 25 Capacity based shear in columns of intermediate frame in XZ plane:

Column No.	Capacity based shear in KN.
15, 105	1.4[(144.41 + 144.41)/ 3.5] = 115.52
45, 75	1.4[(211.24 + 211.24)/ 3.5] = 168.99
14, 104	1.4[(214.07+ 214.07)/ 3.5] = 171.26
44, 74	1.4[(309.63 + 309.63)/ 3.5] = 247.7
13, 103	1.4[(268.83 + 268.83)/ 3.5] = 215.06
43, 73	1.4[(380.68 + 380.68)/ 3.5] = 304.54
12, 102	1.4[(298.69 + 298.69)/ 3.5] = 238.95
42, 72	1.4[(431.99 + 431.99)/ 3.5] = 345.59
11, 101	1.4[(347.33 + 347.33)/ 4.5] = 277.86
41, 71	1.4[(464.51 + 464.51)/ 4.5] = 289.03

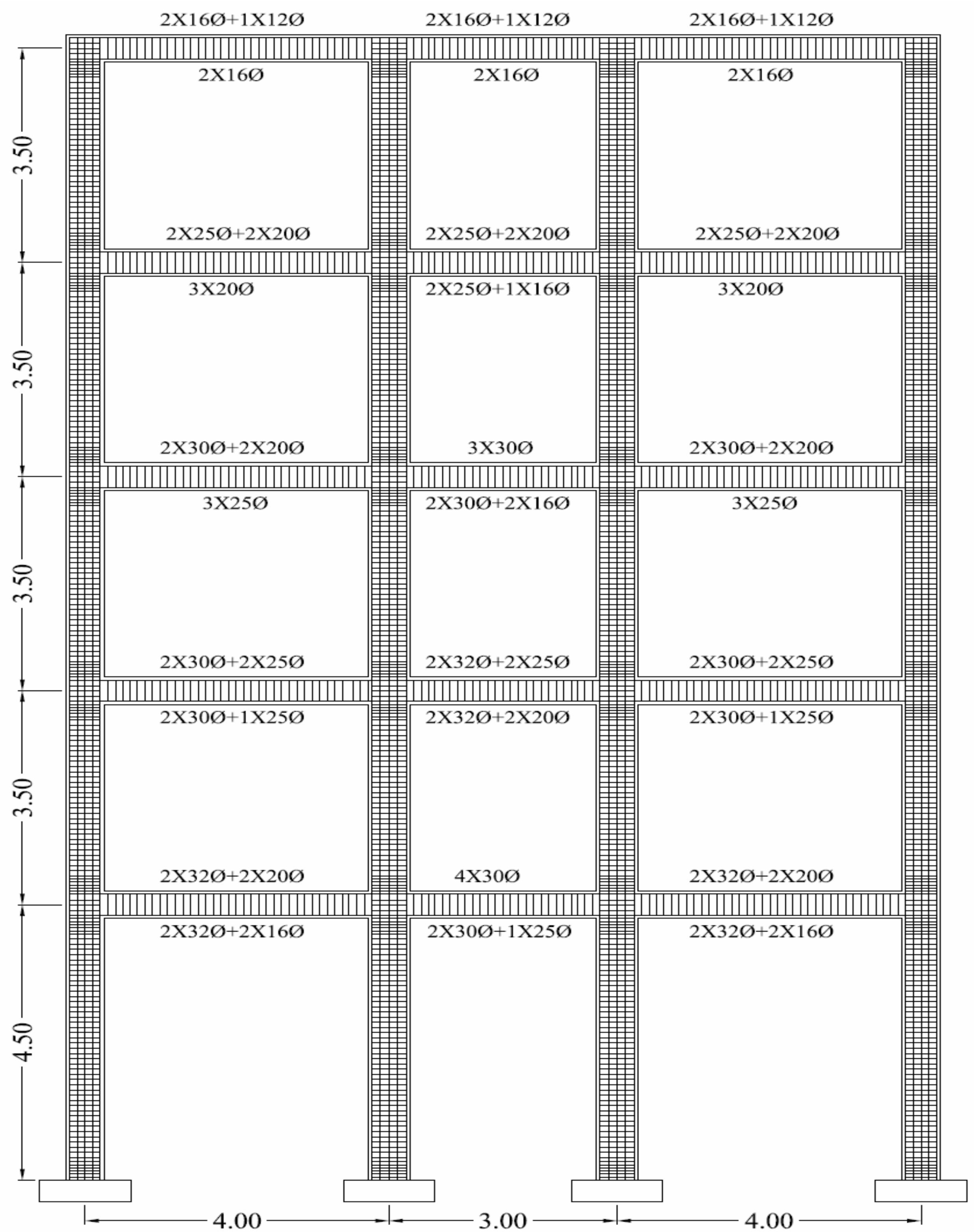
Table 26 Shear reinforcement and special confining reinforcement for columns of intermediate frame in XZ plane:

Column No.	L_0 In mm	Shear reinforcement provided.	Special confining reinforcement provided.
15, 105	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
45, 75	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
14, 104	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
44, 74	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
13, 103	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
43, 73	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
12, 102	510	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
42, 72	550	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.
1, 91	675	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 75mm c/c.
31, 61	675	8Φ two legged stirrup @ 200mm c/c.	10Φ hook @ 80mm c/c.

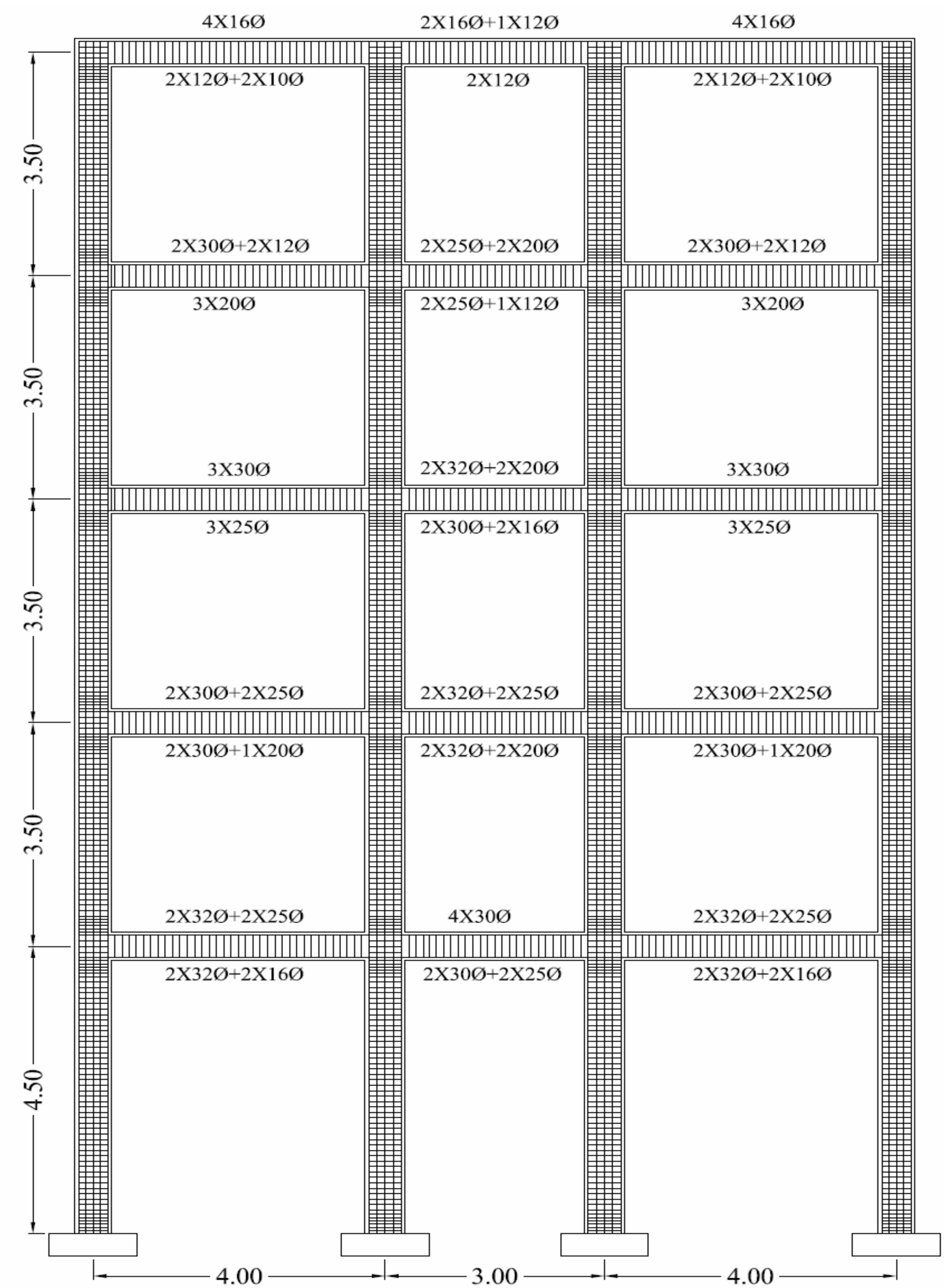
CHAPTER -6

Reinforcement Details

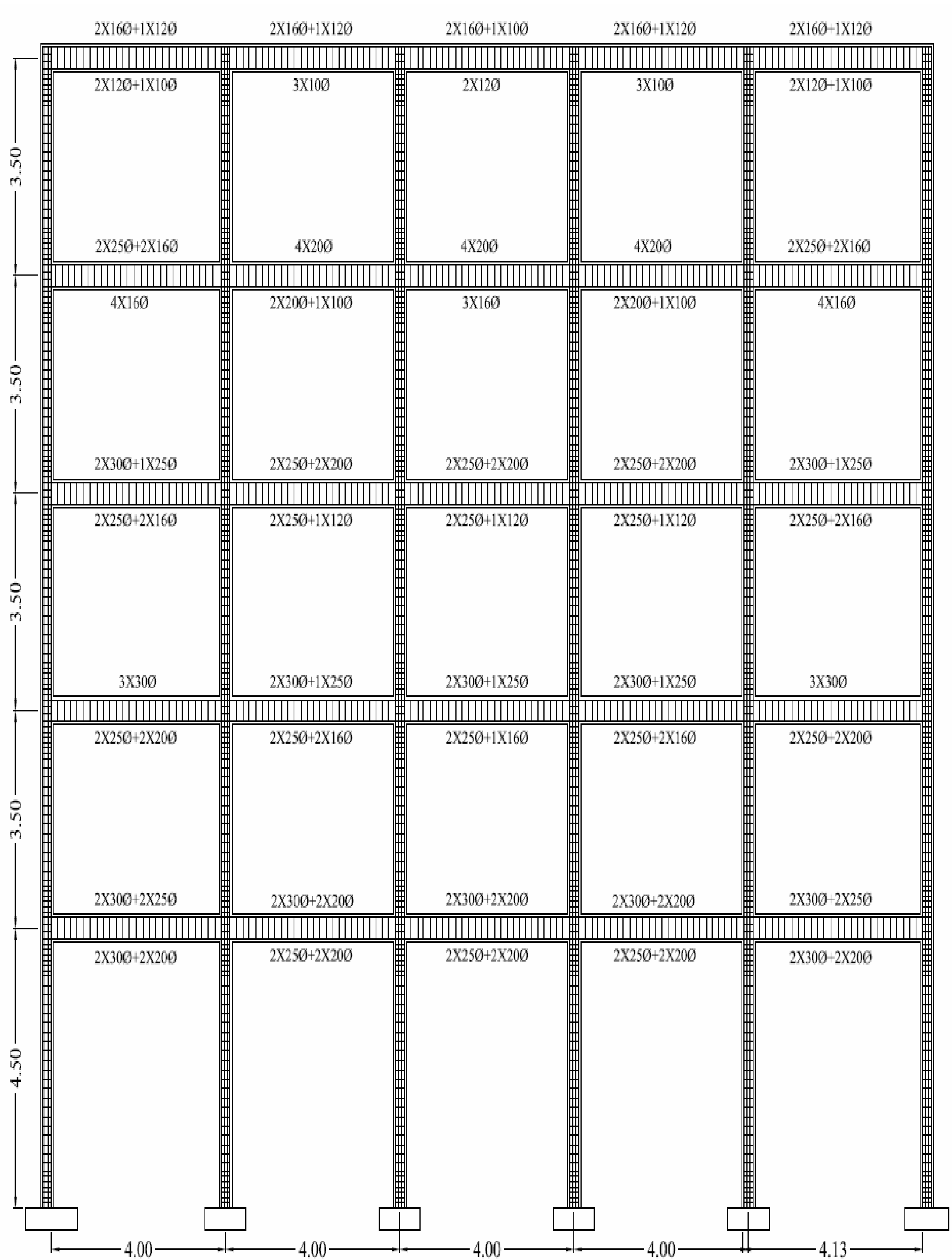
Reinforcement Details of RC Frame:



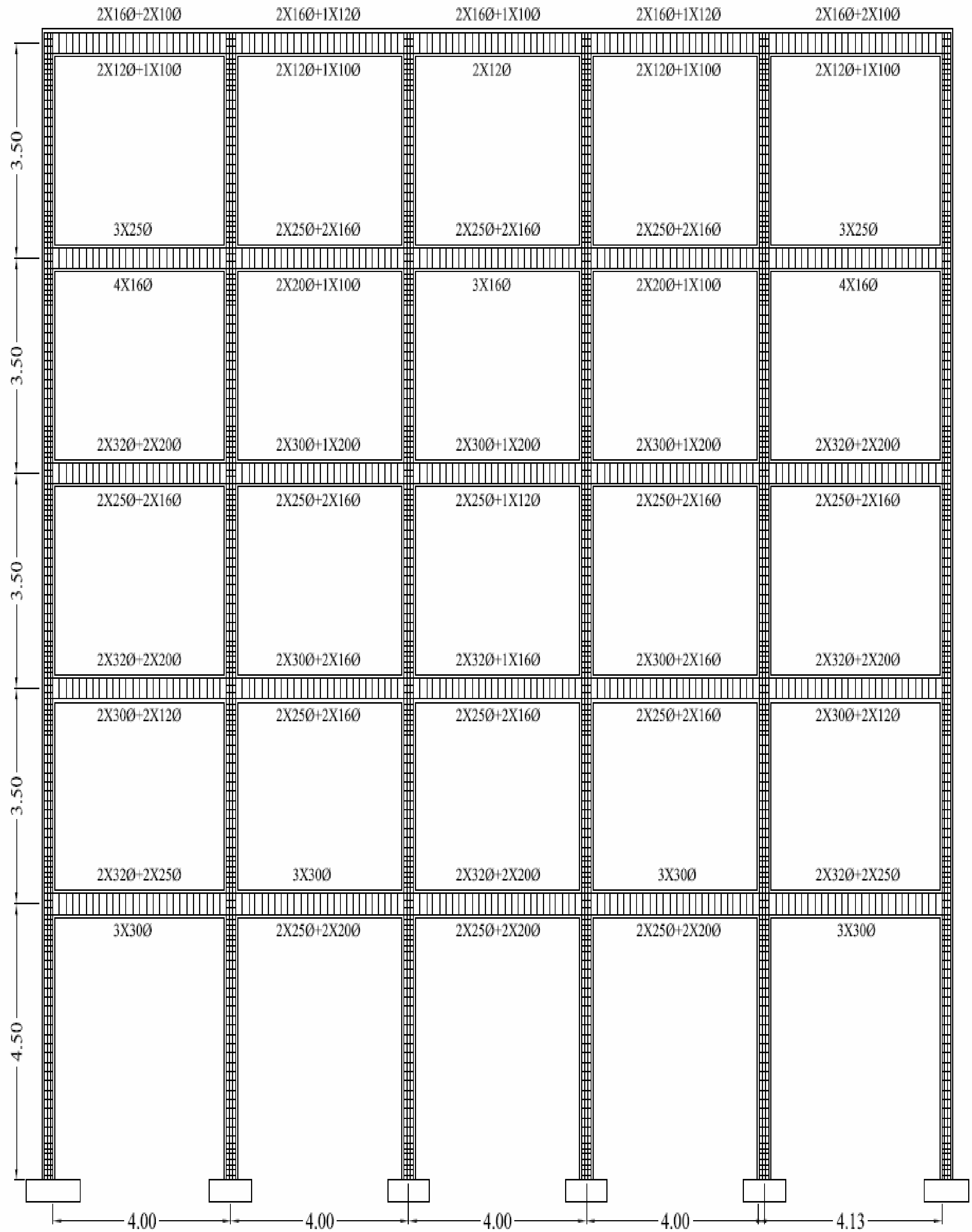
Reinforcement Details of Beams of End Frame in XZ Plane
Figure-46



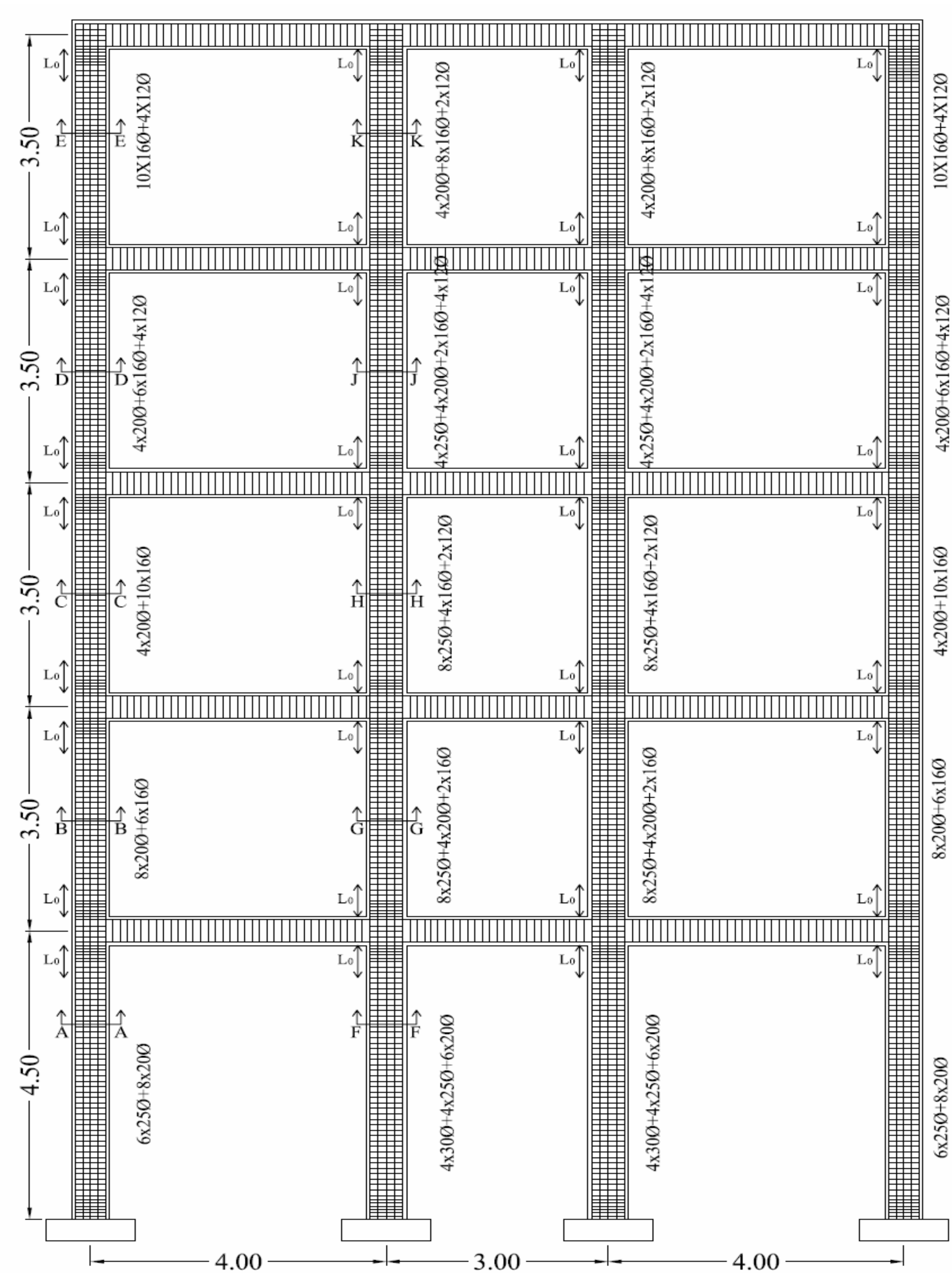
Reinforcement Details of Beams of Intermediate Frame in XZ Plane
Figure-47



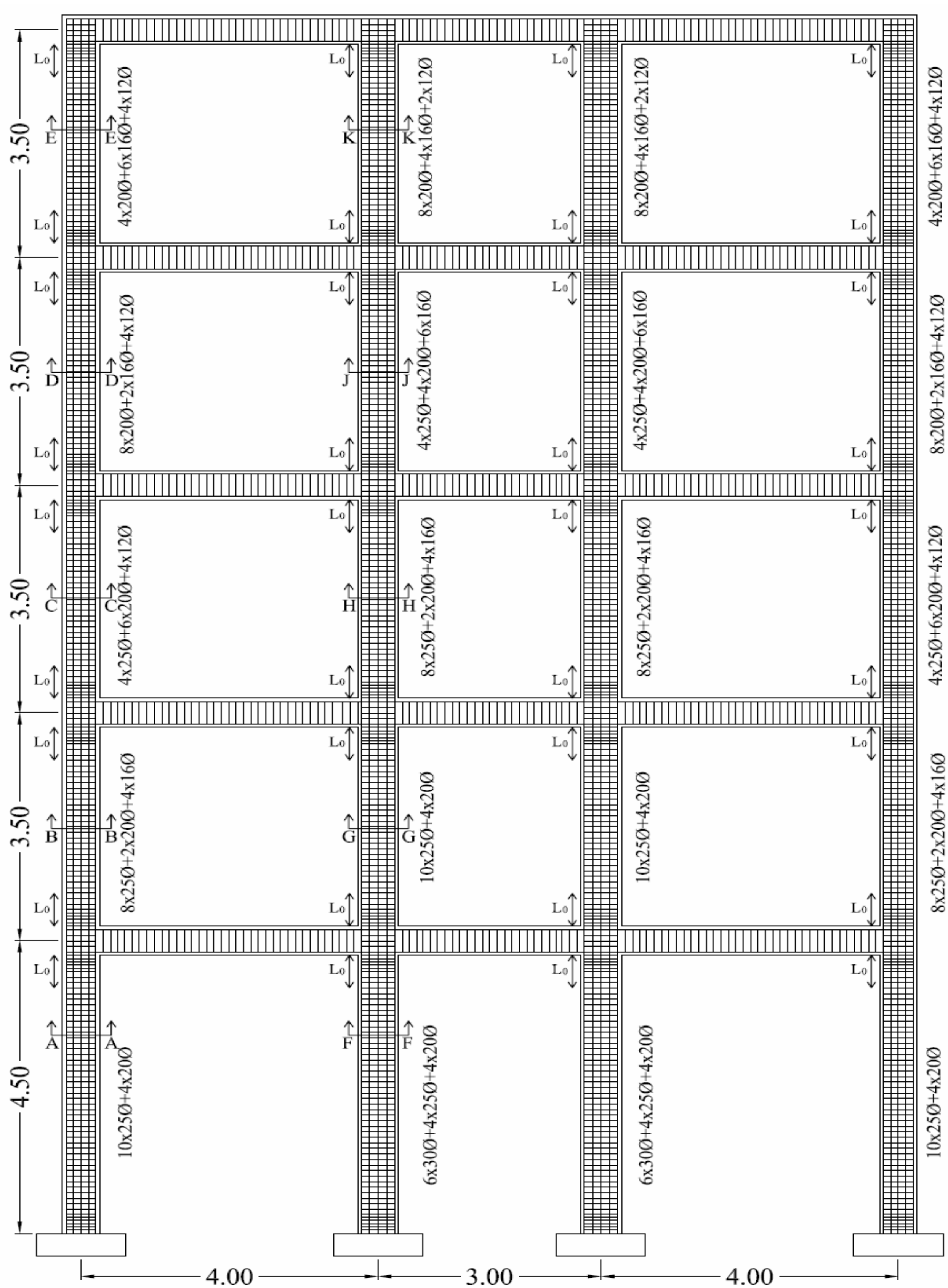
Reinforcement Details of Beams of End Frame in YZ Plane
Figure-48



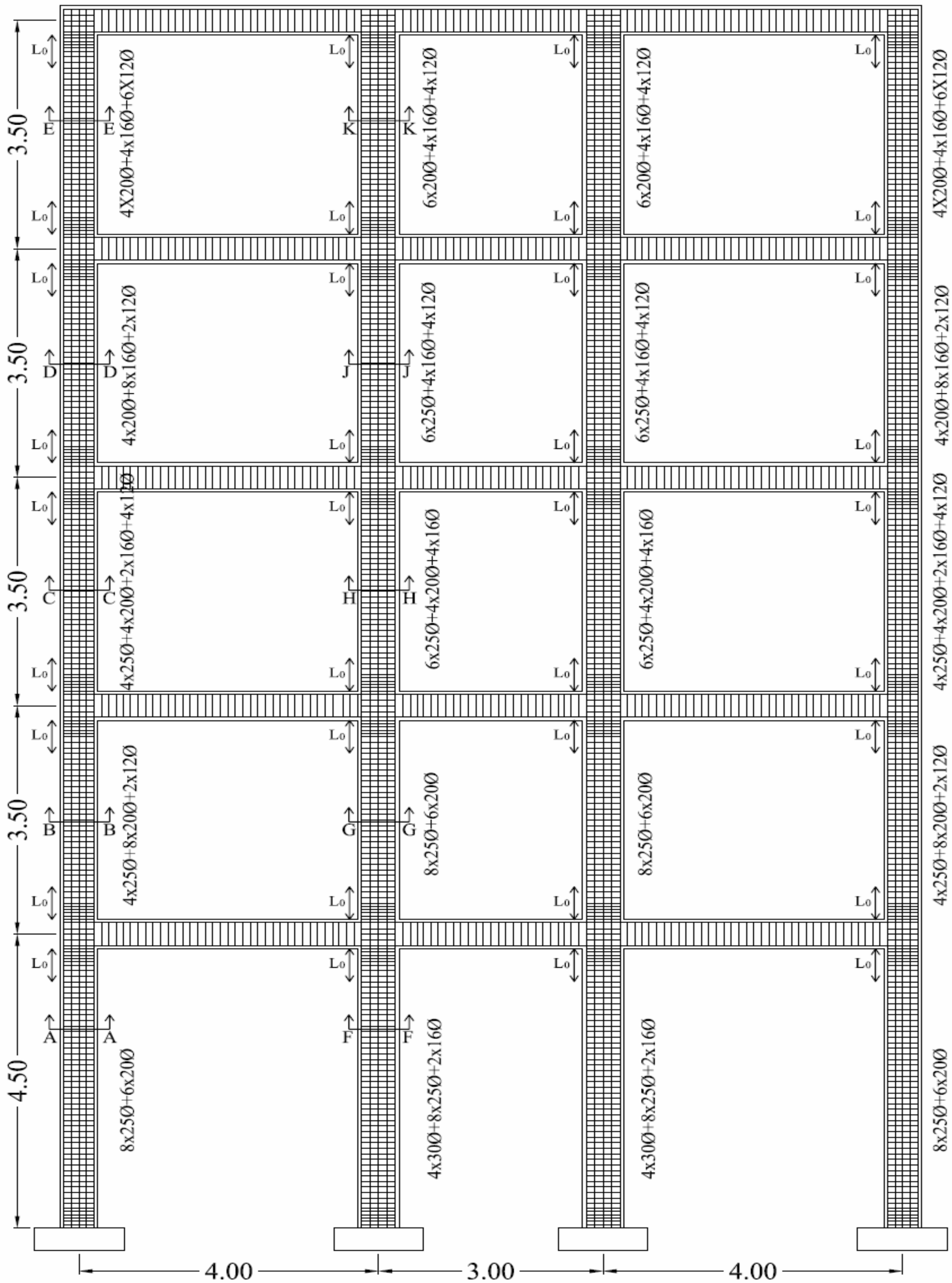
Reinforcement Details of Beams of Intermediate Frame in YZ plane
Figure-49



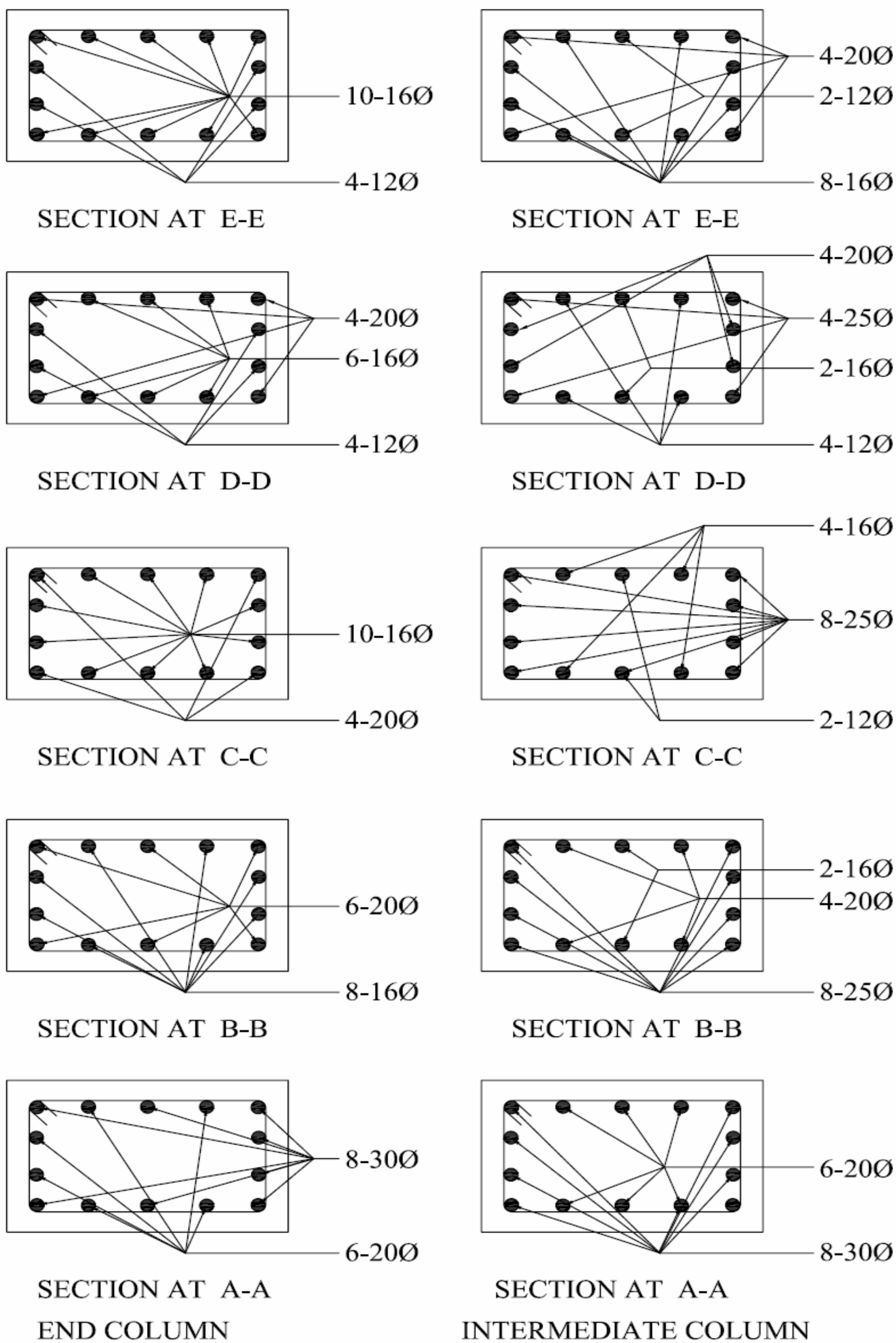
Reinforcement Details of Columns of End Frame in XZ Plane
Figure-50



Reinforcement Details of Columns of Frame Next to End Frame in XZ Plane
Figure-51



Reinforcement Details of Columns of Intermediate Frame in XZ Plane
Figure-52



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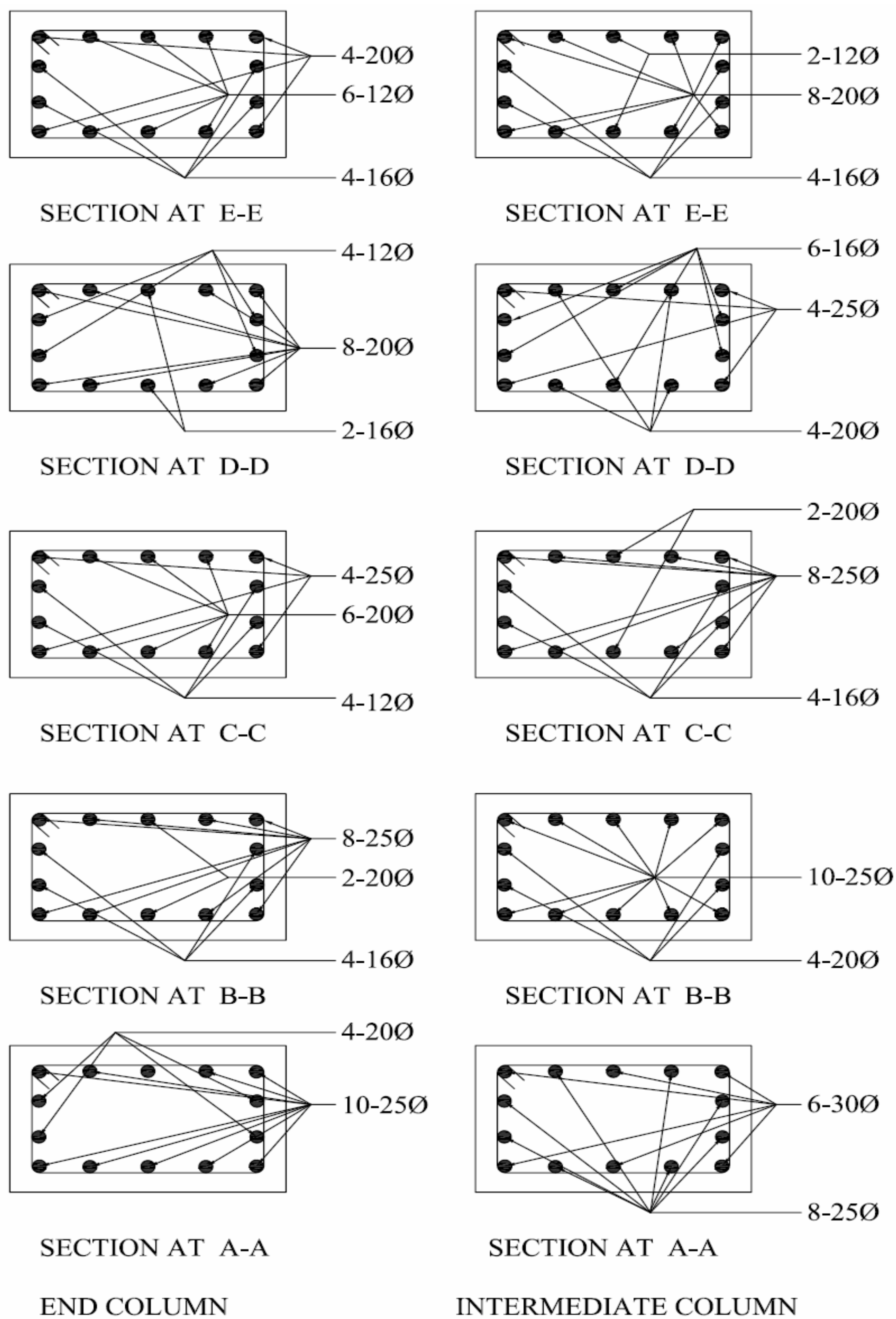
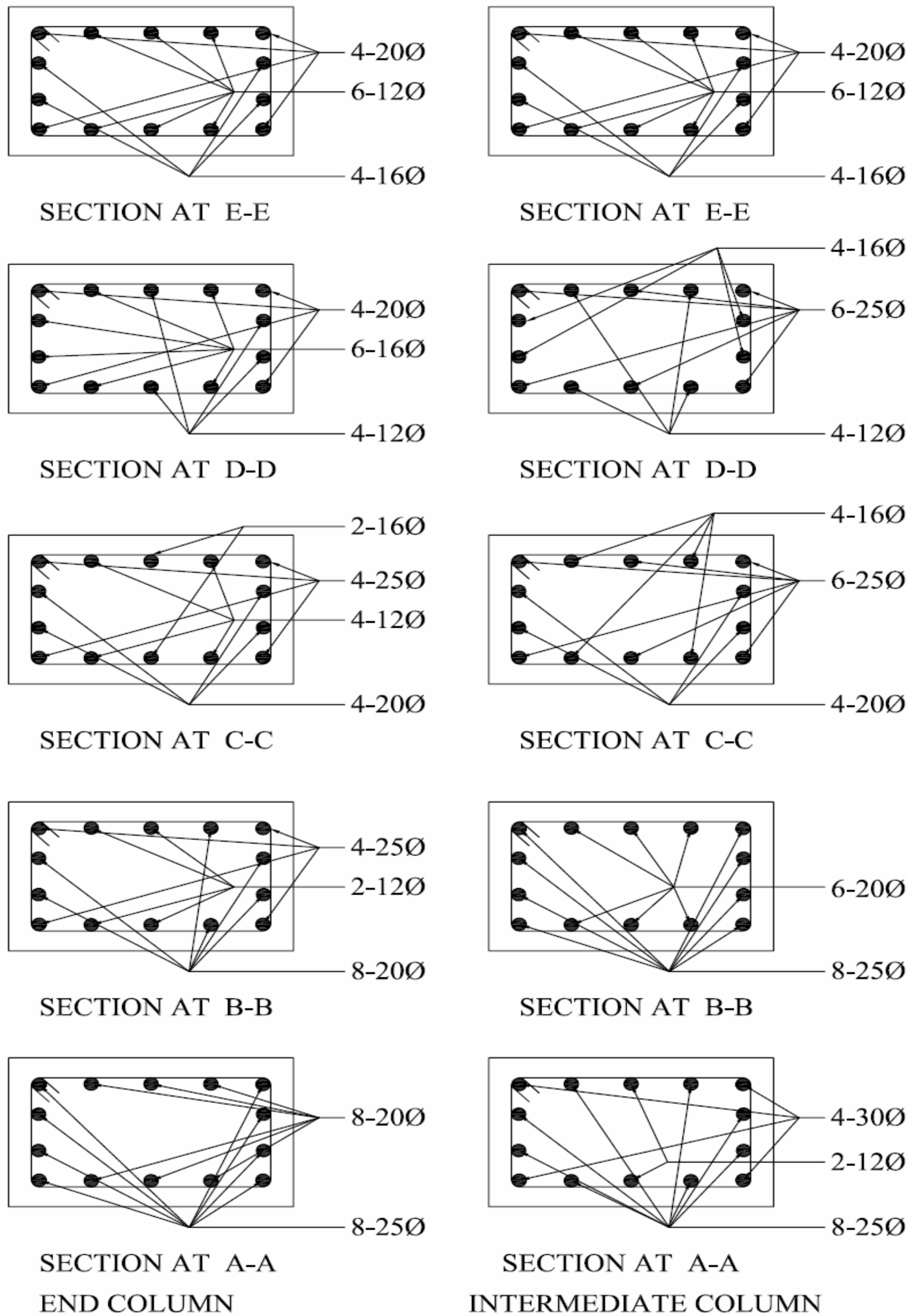
C/S of Columns of End Frame in XZ Plane
Figure-53



C/S of Columns of Frame Next to End Frame XZ Plane
Figure-54



C/S of columns of intermediate frame XZ Plane
Figure-55

CHAPTER - 7

Conclusion

Conclusion

1. Capacity based earthquake resistant design is futuristic approach to design of reinforced concrete structures especially for multi-bay multi storied reinforced concrete buildings.
2. This concept is to restrict the formation of plastic hinges in the beams only hence collapse occurs through the beam mechanism only, which localize the failure and hence leads to less destruction and loss of lives.
3. Collapse due to sway mechanism can cause failure of a storey or whole frame. As its approach is to eliminate sway mechanism by making columns stronger than beams, this method is very effective in design of soft-storey frames.
4. This method also eliminates the possibility of shear mode of failure (which is brittle by nature hence failure occurs suddenly) by making shear capacity of elements more than their moment capacity.
5. Compared with the conventional design methods for earthquake resisting structures although this method is little costlier but is more effective in resisting the earthquake forces.
6. This method of design is more realistic because the calculations are based on provided reinforcement and the over strength of the structure which takes into account the reserve strength beyond elastic limit.
7. As the building can be reused after minimal repairment after occurrence of earthquake hence this method of design should be adopted for public utility buildings like schools, colleges, hospitals etc.

This work can be further verified and checked by performing any non linear analysis on the modified frame.

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Appendix

Appendix - 1

Detail Design of 3D-RC Frame

DESIGN OF FRAME WITH EARTHQUAKE FORCE IN X DIRECTION:

DESIGN OF BEAMS:

Grade of concrete to be used = M 20

Grade of steel to be used = Fe 415

Assuming 25 Φ bar to be used and 25mm clear cover,

Effective depth d = 450 – 25 – (25/2) = 412.5mm.

According to IS 13920:1993

Width / Depth = (300 / 450) = 0.67> 0.3 ok.

Width = 300mm > 200mm. ok.

Depth < (1/4) × Clear span

For end beams:

$$\frac{1}{4}\left(4000-\frac{500}{2}-\frac{550}{2}\right)=868.7mm > 450\text{mm} \quad \text{ok.}$$

For intermediate beams:

$$\frac{1}{4}\left(3000-\frac{550}{2}-\frac{550}{2}\right)=612.5mm > 450\text{mm.} \quad \text{ok.}$$

d^{PI} = 25 + (25/2) = 37.5mm

$$\frac{d'}{d}=\frac{37.5}{412.5}=0.091$$

From table D of SP 16:1980

$$\frac{M_{u\lim}}{bd^2}=2.76 \text{ for M 20 \& Fe 415.}$$

$M_{u\ lim}$ = 2.76 × 300 × $(412.5)^2$

= 140.88 KN.m

<u>Design of beams of end frame in XZ plane:</u>

Beam No. 121, 131:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 320.78 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2}=\frac{320.78\times10^6}{300\times412.5^2}=6.284$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 2.04 \\ P_{bottom} = 1.141 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 250.98 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{250.98 \times 10^6}{300 \times 412.5^2} = 4.917$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.619 \\ P_{bottom} = 0.698 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.04

P_{bottom} = 1.619

% of reinforcement required at top = $\dfrac{2.04 \times 300 \times 412.5}{100} = 2524.5mm^2$

Provide 2 × 32 Φ + 2 × 25 Φ

A_{st} provided = 2590.22 mm^2.

P_{top} provided = 2.093

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 6.455$$

M_u cal top = 329.5 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.619 \times 300 \times 412.5}{100} = 2003.5mm^2$

Provide 2 × 32 Φ + 2 × 16 Φ

A_{st} provided = 2010.66 mm^2.

P_{bottom} provided = 1.625

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.93$$

M_u cal bottom = 251.66 KN.m (Sagging)

Beam No. 122, 132:

For hogging moment:

Maximum hogging moment, $M_u = 289.89$ KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{289.89 \times 10^6}{300 \times 412.5^2} = 5.68$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.855 \\ P_{bottom} &= 0.9454 \end{aligned}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 210.14 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{210.14 \times 10^6}{300 \times 412.5^2} = 4.117$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{bottom} &= 1.373 \\ P_{top} &= 0.4386 \end{aligned}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

$P_{top} = 1.855$

$P_{bottom} = 1.373$

% of reinforcement required at top = $\dfrac{1.855 \times 300 \times 412.5}{100} = 2295.56 mm^2$

Provide $2 \times 30\ \Phi + 2 \times 25\ \Phi$

A_{st} provided = 22395.44 mm^2.

P_{top} provided = 2.1.936

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.945$$

M_u cal top = 303.48 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.373 \times 300 \times 412.5}{100} = 1699.08 mm^2$

Provide 2 × 30 Φ + 1 × 20 Φ

A_{st} provided = 1727.85 mm^2.

P_{bottom} provided = 1.396

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.19$$

M_u cal bottom = 213.89 KN.m (Sagging)

Beam No. 123, 133:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 255.65 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{255.65 \times 10^6}{300 \times 412.5^2} = 5.008$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{top} & = 1.6475 \\ P_{bottom} & = 0.728 \end{array}\right\} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 174.23 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{174.23 \times 10^6}{300 \times 412.5^2} = 3.413$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.156 \\ P_{top} & = 0.211 \end{array}\right\} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.6475

P_{bottom} = 1.156

% of reinforcement required at top = $\dfrac{1.6475 \times 300 \times 412.5}{100} = 2038.78mm^2$

Provide 2 × 30 Φ + 2 × 20 Φ

A_{st} provided = 2042 mm^2.

P_{top} provided = 1.65

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.016$$

M_u cal top = 256.05 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.156 \times 300 \times 412.5}{100} = 1430.55mm^2$

Provide 3 × 25 Φ

A_{st} provided = 1472.61 mm^2.

P_{bottom} provided = 1.19

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.523$$

M_u cal bottom = 179.84 KN.m (Sagging)

Beam No. 124, 134:

For hogging moment:

Maximum hogging moment, M_u = 192.18 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{192.18 \times 10^6}{300 \times 412.5^2} = 3.765$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.265 \\ P_{bottom} &= 0.325 \end{aligned}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 109.27 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{109.27 \times 10^6}{300 \times 412.5^2} = 2.14$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.693 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.265

P_{bottom} = 0.693

% of reinforcement required at top = $\frac{1.265 \times 300 \times 412.5}{100} = 1565.43 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{top} provided = 1.301

Referring to table 50 of SP 16:1980

$\frac{M_u}{bd^2} = 3.883$

M_u cal top = 198.22 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.693 \times 300 \times 412.5}{100} = 857.59 mm^2$

Provide 3 × 20 Φ

A_{st} provided = 942.45 mm^2.

P_{bottom} provided = 0.762

Referring to table 2 of SP 16:1980

$\frac{M_u}{bd^2} = 2.3125$

M_u cal bottom = 118.05 KN.m (Sagging)

Beam No. 125, 135:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 95.11 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$\frac{M_u}{bd^2} = \frac{95.11 \times 10^6}{300 \times 412.5^2} = 1.863$

Referring to table 2 of SP 16:1980

P_{top} = 0.589 — — — — — $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 50.05 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{50.05 \times 10^6}{300 \times 412.5^2} = 0.98$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.289 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.589

P_{bottom} = 0.289

% of reinforcement required at top = $\frac{0.589 \times 300 \times 412.5}{100} = 728.89 mm^2$

Provide 2 × 20 Φ + 1 × 12 Φ

A_{st} provided = 741.39 mm^2.

P_{top} provided = 0.599

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.892$$

M_u cal top = 96.58 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.298 \times 300 \times 412.5}{100} = 357.64 mm^2$

Provide 2 × 16 Φ

A_{st} provided = 402.12 mm^2.

P_{bottom} provided = 0.325

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.094$$

M_u cal bottom = 55.85 KN.m (Sagging)

Beam No. 126:

For hogging moment:

Maximum hogging moment, M_u = 352.79 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{352.79 \times 10^6}{300 \times 412.5^2} = 6.91$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 2.2331 \\ P_{bottom} &= 1.3433 \end{aligned}\right\} \text{— — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 302.91 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{302.91 \times 10^6}{300 \times 412.5^2} = 5.934$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{bottom} &= 1.939 \\ P_{top} &= 1.034 \end{aligned}\right\} \text{— — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.2331

P_{bottom} = 1.939

% of reinforcement required at top = $\dfrac{2.2331 \times 300 \times 412.5}{100} = 2763.46 mm^2$

Provide 4 × 30 Φ

A_{st} provided = 2827.4 mm^2.

P_{top} provided = 2.284

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 7.07$$

M_u cal top = 360.90 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.934 \times 300 \times 412.5}{100} = 2394.56 mm^2$

Provide 2 × 30 Φ + 1 × 25 Φ

A_{st} provided = 2395.44 mm^2.

P_{bottom} provided = 1.936

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.945$$

M_u cal bottom = 303.47 KN.m (Sagging)

Beam No. 127:

For hogging moment:

Maximum hogging moment, M_u = 320.8 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{320.28 \times 10^6}{300 \times 412.5^2} = 6.284$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{top} & = 2.04 \\ P_{bottom} & = 1.141 \end{array}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 274.27 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{392.91 \times 10^6}{300 \times 412.5^2} = 5.373$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.76 \\ P_{top} & = 0.845 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.04

P_{bottom} = 1.76

% of reinforcement required at top = $\dfrac{2.04 \times 300 \times 412.5}{100} = 2524.5mm^2$

Provide 2 × 32 Φ + 2 × 25 Φ

A_{st} provided = 2590.22 mm^2.

P_{top} provided = 2.2.093

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 6.455$$

M_u cal top = 329.5 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.76 \times 300 \times 412.5}{100} = 2178mm^2$

Provide 2 × 32 Φ + 2 × 20 Φ

A_{st} provided = 2236.76 mm^2.

P_{bottom} provided = 1.807

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.526$$

M_u cal bottom = 282.09 KN.m (Sagging)

Beam No. 128:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 263.77 KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{263.77 \times 10^6}{300 \times 412.5^2} = 5.167$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.697 \\ P_{bottom} = 0.779 \end{array}\right\} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 216.86 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{216.86 \times 10^6}{300 \times 412.5^2} = 4.248$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.4134 \\ P_{top} = 0.481 \end{array}\right\} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.697

P_{bottom} = 1.4134

% of reinforcement required at top = $\dfrac{1.697 \times 300 \times 412.5}{100} = 2100.04 mm^2$

Provide 3 × 30 Φ

A_{st} provided = 2120.55 mm^2.

P_{top} provided = 1.714

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.223$$

M_u cal top = 266.62 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.4134 \times 300 \times 412.5}{100} = 1749.08 mm^2$

Provide 2 × 30 Φ + 2 × 16 Φ

A_{st} provided = 1815.82 mm^2.

P_{bottom} provided = 1.4673

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.4235$$

M_u cal bottom = 225.81 KN.m (Sagging)

Beam No. 129:

For hogging moment:

Maximum hogging moment, M_u = 180.18 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{180.18 \times 10^6}{300 \times 412.5^2} = 3.53$$

Referring to table 50 of SP 16:1980

P_{top} = 1.1923

P_{bottom} = 0.249 — — — — — (1)

For sagging moment:

Maximum sagging moment = 134.23 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{134.23 \times 10^6}{300 \times 412.5^2} = 2.63$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.897 — — — — — (2)

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.1923

P_{bottom} = 0.897

% of reinforcement required at top = $\frac{1.1923 \times 300 \times 412.5}{100} = 1475.47 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{top} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal top = 198.22 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.897 \times 300 \times 412.5}{100} = 1110.03 mm^2$

Provide 2 × 25 Φ + 1 × 16 Φ

A_{st} provided = 1182.8 mm^2.

P_{bottom} provided = 0.955

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.76$$

M_u cal bottom = 140.89 KN.m (Sagging)

Beam No. 130:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 64.99 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{64.99 \times 10^6}{300 \times 412.5^2} = 1.273$$

Referring to table 2 of SP 16:1980

P_{top} = 0.383 — — — — — $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 46.11 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{46.11 \times 10^6}{300 \times 412.5^2} = 0.903$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.265 —— —— —— —— —— $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.383

P_{bottom} = 0.266

% of reinforcement required at top = $\frac{0.383 \times 300 \times 412.5}{100} = 473.96 mm^2$

Provide 2 × 16 Φ + 1 × 12 Φ

A_{st} provided = 515.21 mm^2.

P_{top} provided = 0.416

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.371$$

M_u cal top = 69.99 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.266 \times 300 \times 412.5}{100} = 329.17 mm^2$

Provide 2 × 16 Φ

A_{st} provided = 402.12 mm^2.

P_{bottom} provided = 0.325

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.094$$

M_u cal bottom = 55.85 KN.m (Sagging)

<u>Design of beams of intermediate frame in XZ plane:</u>

Beam No. 136, 146:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 329 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{329 \times 10^6}{300 \times 412.5^2} = 6.445$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 2.08 \\ P_{bottom} = 1.19 \end{array}\right\} \text{— — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 247.82 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{247.82 \times 10^6}{300 \times 412.5^2} = 4.85$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.59 \\ P_{bottom} = 0.676 \end{array}\right\} \text{— — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.08

P_{bottom} = 1.59

% of reinforcement required at top = $\frac{2.08 \times 300 \times 412.5}{100} = 2574 mm^2$

Provide 2 × 32 Φ + 2 × 25 Φ

A_{st} provided = 2590.22 mm^2.

P_{top} provided = 2.093

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 6.455$$

M_u cal top = 329.5 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.59 \times 300 \times 412.5}{100} = 1967.62 mm^2$

Provide 2 × 32 Φ + 2 × 16 Φ

A_{st} provided = 2010.66 mm^2.

P_{bottom} provided = 1.625

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.93$$

M_u cal bottom = 251.66 KN.m (Sagging)

Beam No. 137, 147:

<u>For hogging moment:</u>

Maximum hogging moment, $M_u = 299.07$ KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{299.07 \times 10^6}{300 \times 412.5^2} = 5.86$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{top} & = 1.91 \\ P_{bottom} & = 1.003 \end{array}\right\} \text{-------} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 206.26 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{206.26 \times 10^6}{300 \times 412.5^2} = 4.04$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.349 \\ P_{top} & = 0.414 \end{array}\right\} \text{-------} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.91

P_{bottom} = 1.349

% of reinforcement required at top = $\dfrac{1.91 \times 300 \times 412.5}{100} = 2363.62 mm^2$

Provide 2 × 30 Φ + 2 × 25 Φ

A_{st} provided = 22395.44 mm^2.

P_{top} provided = 2.1.936

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.945$$

M_u cal top = 303.48 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.349 \times 300 \times 412.5}{100} = 1669.38 mm^2$

Provide 2 × 30 Φ + 1 × 20 Φ

A_{st} provided = 1727.85 mm^2.

P_{bottom} provided = 1.396

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.19$$

M_u cal bottom = 213.89 KN.m (Sagging)

Beam No. 138, 148:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 265.06 KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{265.06 \times 10^6}{300 \times 412.5^2} = 5.19$$

Referring to table 50 of SP 16:1980

P_{top} = 1.703

P_{bottom} = 0.787 — — — — — $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 170.07 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{170.07 \times 10^6}{300 \times 412.5^2} = 3.34$$

Referring to table 50 of SP 16:1980

P_{bottom} = 1.134

P_{top} = 0.187 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.703

P_{bottom} = 1.134

% of reinforcement required at top = $\frac{1.703 \times 300 \times 412.5}{100} = 2107.46 mm^2$

Provide 3 × 30 Φ

A_{st} provided = 2120.55 mm^2.

P_{top} provided = 1.714

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.223$$

M_u cal top = 266.62 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.134 \times 300 \times 412.5}{100} = 1403.33 mm^2$

Provide 3 × 25 Φ

A_{st} provided = 1472.61 mm^2.

P_{bottom} provided = 1.19

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.523$$

M_u cal bottom = 179.84 KN.m (Sagging)

Beam No. 139, 149:

For hogging moment:

Maximum hogging moment, M_u = 201.25 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{201.25 \times 10^6}{300 \times 412.5^2} = 3.94$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.318 \\ P_{bottom} &= 0.382 \end{aligned}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 106.43 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{106.43 \times 10^6}{300 \times 412.5^2} = 2.085$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.672 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.318

P_{bottom} = 0.672

% of reinforcement required at top = $\frac{1.318 \times 300 \times 412.5}{100} = 1631.03 mm^2$

Provide 2 × 30 Φ + 2 × 12 Φ

A_{st} provided = 1639.88 mm^2.

P_{top} provided = 1.325

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.96$$

M_u cal top = 202.14 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.672 \times 300 \times 412.5}{100} = 831.6 mm^2$

Provide 3 × 20 Φ

A_{st} provided = 942.45 mm^2.

P_{bottom} provided = 0.762

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.3125$$

M_u cal bottom = 118.05 KN.m (Sagging)

Beam No. 140, 150:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 96.92 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{96.92 \times 10^6}{300 \times 412.5^2} = 1.898$$

Referring to table 2 of SP 16:1980

P_{top} = 0.601 — — — — — $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 50.55 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{50.55 \times 10^6}{300 \times 412.5^2} = 0.99$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.292 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.601

P_{bottom} = 0.292

% of reinforcement required at top = $\frac{0.601 \times 300 \times 412.5}{100} = 743.74 mm^2$

Provide 4 × 16 Φ

A_{st} provided = 804.24 mm^2.

P_{top} provided = 0.649

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.025$$

M_u cal top = 103.57 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.292 \times 300 \times 412.5}{100} = 361.35 mm^2$

Provide 2 × 12 Φ + 2 × 10 Φ

A_{st} provided = 383.24 mm^2.

P_{bottom} provided = 0.31

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.047$$

M_u cal bottom = 53.44 KN.m (Sagging)

Beam No. 141:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 358.02 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{358.08 \times 10^6}{300 \times 412.5^2} = 7.01$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{top} & = 2.264 \\ P_{bottom} & = 1.376 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 302.83 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{302.84 \times 10^6}{300 \times 412.5^2} = 5.93$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.931 \\ P_{top} & = 1.026 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.264

P_{bottom} = 1.931

% of reinforcement required at top = $\dfrac{2.264 \times 300 \times 412.5}{100} = 2801.7 mm^2$

Provide 4 × 30 Φ

A_{st} provided = 2827.4 mm^2.

P_{top} provided = 2.284

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 7.07$$

M_u cal top = 360.90 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.931 \times 300 \times 412.5}{100} = 2389.61 mm^2$

Provide 2 × 30 Φ + 1 × 25 Φ

A_{st} provided = 2395.44 mm^2.

P_{bottom} provided = 1.936

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.945$$

M_u cal bottom = 303.47 KN.m (Sagging)

Beam No. 142:

For hogging moment:

Maximum hogging moment, $M_u = 325.73$ KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{325.73 \times 10^6}{300 \times 412.5^2} = 6.38$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{top} & = 2.07 \\ P_{bottom} & = 1.122 \end{array}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 273.38 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{273.38 \times 10^6}{300 \times 412.5^2} = 5.355$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.754 \\ P_{top} & = 0.839 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.07

P_{bottom} = 1.754

% of reinforcement required at top = $\dfrac{2.07 \times 300 \times 412.5}{100} = 2561.62 mm^2$

Provide 2 × 32 Φ + 2 × 25 Φ

A_{st} provided = 2590.22 mm^2.

P_{top} provided = 2.2.093

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 6.455$$

M_u cal top = 329.5 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.754 \times 300 \times 412.5}{100} = 2170.58 mm^2$

Provide 2 × 32 Φ + 2 × 20 Φ

A_{st} provided = 2236.76 mm^2.

P_{bottom} provided = 1.807

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.526$$

M_u cal bottom = 282.09 KN.m (Sagging)

Beam No. 143:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 268.46 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{268.48 \times 10^6}{300 \times 412.5^2} = 5.259$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.7247 \\ P_{bottom} &= 0.809 \end{aligned}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 215.76 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{215.76 \times 10^6}{300 \times 412.5^2} = 4.227$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{bottom} &= 1.404 \\ P_{top} &= 0.475 \end{aligned}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.7247

P_{bottom} = 1.404

% of reinforcement required at top = $\frac{1.7247 \times 300 \times 412.5}{100} = 2134.42 mm^2$

Provide 2 × 32 Φ + 2 × 20 Φ

A_{st} provided = 2236.76 mm^2.

P_{top} provided = 1.807

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.477$$

M_u cal top = 282.09 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.407 \times 300 \times 412.5}{100} = 1741.16 mm^2$

Provide 2 × 30 Φ + 2 × 16 Φ

A_{st} provided = 1815.82 mm^2.

P_{bottom} provided = 1.4673

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.4235$$

M_u cal bottom = 225.81 KN.m (Sagging)

Beam No. 144:

For hogging moment:

Maximum hogging moment, M_u = 185.05 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{185.05 \times 10^6}{300 \times 412.5^2} = 3.625$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.222 \\ P_{bottom} = 0.279 \end{array}\right\} \text{— — — — — } \boxed{1}$$

For sagging moment:

Maximum sagging moment = 132.58 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{132.58 \times 10^6}{300 \times 412.5^2} = 2.597$$

Referring to table 2 of SP 16:1980

$$P_{bottom} = 0.882 \text{ — — — — — } \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.222

P_{bottom} = 0.882

% of reinforcement required at top = $\frac{1.222 \times 300 \times 412.5}{100} = 1512.22 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{top} provided = 1.301

Referring to table 50 of SP 16:1980

$\frac{M_u}{bd^2} = 3.883$

M_u cal top = 198.22 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.882 \times 300 \times 412.5}{100} = 1091.42 mm^2$

Provide 2 × 25 Φ + 1 × 12 Φ

A_{st} provided = 1094.83 mm^2.

P_{bottom} provided = 0.8847

Referring to table 2 of SP 16:1980

$\frac{M_u}{bd^2} = 2.604$

M_u cal bottom = 132.93 KN.m (Sagging)

Beam No. 145:

For hogging moment:

Maximum hogging moment, M_u = 68.75 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$\frac{M_u}{bd^2} = \frac{68.75 \times 10^6}{300 \times 412.5^2} = 1.347$

Referring to table 2 of SP 16:1980

P_{top} = 0.408 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 44.37 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{44.37 \times 10^6}{300 \times 412.5^2} = 0.869$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.254 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.408

P_{bottom} = 0.254

% of reinforcement required at top = $\frac{0.408 \times 300 \times 412.5}{100} = 504.9mm^2$

Provide 2 × 16 Φ + 1 × 12 Φ

A_{st} provided = 515.21 mm^2.

P_{top} provided = 0.416

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.371$$

M_u cal top = 69.99 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.254 \times 300 \times 412.5}{100} = 314.32mm^2$

Provide 2 × 12 Φ

A_{st} provided = 339.27 mm^2.

P_{bottom} provided = 0.274

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.931$$

M_u cal bottom = 47.52 KN.m (Sagging)

<u>Design of columns of end frame in XZ plane:</u>

Column No. 5, 95:

Axial force, P_u = 126.12KN

Axial stress = $\frac{126.12 \times 10^3}{400 \times 500} = 0.63 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 126.12 × 0.02277 = 2.87KN.m

M_{uy} from analysis = 27.5KN.m

So M_{uy} = 27.5KN.m

M_{ux} = 135.06 KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{126.12 \times 10^3}{20 \times 400 \times 500} = 0.032$$

Take percentage of steel, p = 1.1%

$$\frac{p}{f_{ck}} = \frac{1.1}{20} = 0.055$$

For M_{ux1}:

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.088$$

$$\therefore M_{ux1} = 0.088 \times 20 \times 400 \times 500^2 = 176KN.m$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.079$$

$$\therefore M_{uy1} = 0.079 \times 20 \times 500 \times 400^2 = 126.4KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.1%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 11.5$$

$P_{uz} = 11.5 \times 400 \times 500 = 2300KN$

$$\frac{P_u}{P_{uz}} = \frac{126.12}{2300} = 0.055 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{135.05}{176}\right)^1 + \left(\frac{27.05}{126.4}\right)^1 = 0.985$$

Column No. 35, 65:

Axial force, $P_u = 143.92KN$

Axial stress = $\frac{143.92 \times 10^3}{400 \times 550} = 0.65 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P|} = 40 + 10 = 50mm.$

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So $e_{min} = 24.43mm.$

$M_{uy} = 143.92 \times 0.02443 = 3.52KN.m$

M_{uy} from analysis = 34.28KN.m

So $M_{uy} = 34.28KN.m$

$M_{ux} = 213.34KN.m$

$$\frac{P_u}{f_{ck}bd} = \frac{143.92 \times 10^3}{20 \times 400 \times 550} = 0.033$$

Take percentage of steel, p = 1.125%

$$\frac{p}{f_{ck}} = \frac{1.125}{20} = 0.05625$$

For M_{ux1}:

b = 400mm,   D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.11$$

$$\therefore M_{ux1} = 0.11 \times 20 \times 400 \times 550^2 = 266.2KN.m$$

For M_{uy1}:

b = 550mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.099$$

$$\therefore M_{uy1} = 0.099 \times 20 \times 550 \times 400^2 = 174.24KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.125%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 11.8$$

P_{uz} = 11.8 × 400 × 550 = 2596KN

$$\frac{P_u}{P_{uz}} = \frac{143.92}{2596} = 0.055 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{213.34}{266.2}\right)^1 + \left(\frac{34.28}{174.24}\right)^1 = 0.998$$

Column No. 4, 94:

Axial force, P_u = 342.42KN

Axial stress = $\frac{342.42 \times 10^3}{400 \times 500} = 1.172 < 0.1f_{ck} = 2$ N/ mm^2

Assuming to use 20Φ bar with a clear cover 40mm

d^{P1} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 342.42 × 0.02277 = 7.7KN.m

M_{uy} from analysis = 25.2KN.m

So M_{uy} = 25.2KN.m

M_{ux} = 168.43 KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{342.42 \times 10^3}{20 \times 400 \times 500} = 0.086$$

Take percentage of steel, p = 1.15%

$$\frac{p}{f_{ck}} = \frac{1.15}{20} = 0.0575$$

For M_{ux1}:

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.102$$

$$\therefore M_{ux1} = 0.102 \times 20 \times 400 \times 500^2 = 204KN.m$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.093$$

$$\therefore M_{uy1} = 0.093 \times 20 \times 500 \times 400^2 = 148.8KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.15%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 12.5$$

$P_{uz} = 12.5 \times 400 \times 500 = 2500KN$

$$\frac{P_u}{P_{uz}} = \frac{342..42}{2500} = 0.137 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{168.43}{204}\right)^1 + \left(\frac{25.2}{148.8}\right)^1 = 0.995$$

Column No. 34, 64:

Axial force, $P_u = 383.26KN$

Axial stress = $\frac{383.26 \times 10^3}{400 \times 550} = 1.74 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50mm.$

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So $e_{min} = 24.43mm$.

$M_{uy} = 383.26 \times 0.02443 = 9.36KN.m$

M_{uy} from analysis = 32.49KN.m

So $M_{uy} = 32.49KN.m$

$M_{ux} = 300.75KN.m$

$$\frac{P_u}{f_{ck}bd} = \frac{383.26 \times 10^3}{20 \times 400 \times 550} = 0.087$$

Take percentage of steel, p = 1.625%

$$\frac{p}{f_{ck}} = \frac{1.625}{20} = 0.08125$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.146$$

$$\therefore M_{ux1} = 0.146 \times 20 \times 400 \times 550^2 = 353.32 KN.m$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.129$$

$$\therefore M_{uy1} = 0.129 \times 20 \times 550 \times 400^2 = 227.04 KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.625%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 14.8$$

$P_{uz} = 14.8 \times 400 \times 550 = 3256$KN

$$\frac{P_u}{P_{uz}} = \frac{383.26}{3256} = 0.177 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{300.75}{353.32}\right)^1 + \left(\frac{32.49}{227.04}\right)^1 = 0.994$$

Column No. 3, 93:

Axial force, P_u = 589.27KN

Axial stress = $\frac{589.27 \times 10^3}{400 \times 500} = 2.94 > 0.1 f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4 \qquad \text{ok.}$$

Assuming to use 20Φ bar with a clear cover 40mm

$d^{PI} = 40 + 10 = 50$mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 589.27 × 0.02277 = 13.42KN.m

M_{uy} from analysis = 24.14KN.m

So M_{uy} = 24.14KN.m

M_{ux} = 210.22KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{589.27 \times 10^3}{20 \times 400 \times 500} = 0.147$$

Take percentage of steel, p = 1.225%

$$\frac{p}{f_{ck}} = \frac{1.225}{20} = 0.06125$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.121$$

$$\therefore M_{ux1} = 0.121 \times 20 \times 400 \times 500^2 = 242KN.m$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.111$$

$$\therefore M_{uy1} = 0.111 \times 20 \times 500 \times 400^2 = 177.6KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.225%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.2$$

P_{uz} = 13.2 × 400 × 500 = 2640KN

$$\frac{P_u}{P_{uz}} = \frac{589.27}{2640} = 0.223 > 0.2$$

$\therefore \alpha_n = 1.038$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{210.22}{242}\right)^{1.038} + \left(\frac{24.14}{177.6}\right)^{1.038} = 0.99$$

Column No. 33, 63:

Axial force, P_u = 622.68KN

Axial stress = $\frac{622.68 \times 10^3}{400 \times 550}$ = 2.83 > $0.1f_{ck}$ = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{uy} = 622.68 × 0.02443 = 15.21KN.m

M_{uy} from analysis = 30.996KN.m

So $M_{uy} = 30.996KN.m$

$M_{ux} = 380.17KN.m$

$$\frac{P_u}{f_{ck}bd} = \frac{622.68 \times 10^3}{20 \times 400 \times 550} = 0.142$$

Take percentage of steel, p = 1.875%

$$\frac{p}{f_{ck}} = \frac{1.875}{20} = 0.09375$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.177$$

$$\therefore M_{ux1} = 0.177 \times 20 \times 400 \times 550^2 = 428.34KN.m$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.157$$

$$\therefore M_{uy1} = 0.157 \times 20 \times 550 \times 400^2 = 276.32KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.875%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 16.6$$

$P_{uz} = 16.6 \times 400 \times 550 = 3652KN$

$$\frac{P_u}{P_{uz}} = \frac{622.68}{3652} = 0.171 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{380.17}{428.34}\right)^1 + \left(\frac{30.996}{276.32}\right)^1 = 0.999$$

Column No. 2, 92:

Axial force, P_u = 852.14KN

Axial stress = $\frac{852.14\times10^3}{400\times500} = 4.26 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P|}$ = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05\times10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 852.14 × 0.02277 = 19.4KN.m

M_{uy} from analysis = 25.44KN.m

So M_{uy} = 25.44KN.m

M_{ux} = 214.77KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{852.14\times10^3}{20\times400\times500} = 0.213$$

Take percentage of steel, p = 1.45%

$$\frac{p}{f_{ck}} = \frac{1.45}{20} = 0.0725$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.119$$

$$\therefore M_{ux1} = 0.119 \times 20 \times 400 \times 500^2 = 238KN.m$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.105$$

$$\therefore M_{uy1} = 0.105 \times 20 \times 500 \times 400^2 = 168KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.45%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.22$$

$P_{uz} = 13.22 \times 400 \times 500 = 2644KN$

$$\frac{P_u}{P_{uz}} = \frac{852.14}{2644} = 0.322 > 0.2$$

$\therefore \alpha_n = 1.2$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{214.77}{238}\right)^{1.2} + \left(\frac{25.44}{168}\right)^{1.2} = 0.988$$

Column No. 32, 62:

Axial force, P_u = 885.6KN

Axial stress = $\frac{885.6 \times 10^3}{400 \times 550} = 4.03 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50mm.$

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{uy} = 885.6 × 0.02443 = 24.53KN.m

M_{uy} from analysis = 32.82KN.m

So M_{uy} = 38.82KN.m

M_{ux} = 430.62KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{885.6 \times 10^3}{20 \times 400 \times 550} = 0.198$$

Take percentage of steel, p = 2.4%

$$\frac{p}{f_{ck}} = \frac{2.4}{20} = 0.12$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.201$$

$$\therefore M_{ux1} = 0.201 \times 20 \times 400 \times 550^2 = 479.16KN.m$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.176$$

$$\therefore M_{uy1} = 0.176 \times 20 \times 550 \times 400^2 = 309.76KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.4%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 17.9$$

P_{uz} = 17.9 × 400 × 550 = 3938KN

$$\frac{P_u}{P_{uz}} = \frac{885.6}{3938} = 0.24 > 0.2$$

$\therefore \alpha_n = 1.042$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{430.62}{479.16}\right)^{1.042} + \left(\frac{32.82}{309.76}\right)^{1.042} = 0.991$$

Column No. 1, 91:

Axial force, P_u = 1131.58KN

Axial stress = $\frac{1131.58 \times 10^3}{400 \times 500} = 5.66 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4 \qquad \text{ok.}$$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{500}{30} = 24.77mm$$

or 20mm which ever is greater.

So e_{min} = 24.77mm.

M_{uy} = 1131.58 × 0.02477 = 28.02KN.m

M_{uy} from analysis = 13.29KN.m

So M_{uy} = 28.02KN.m

M_{ux} = 214.77KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1131.58 \times 10^3}{20 \times 400 \times 500} = 0.283$$

Take percentage of steel, p = 2.15%

$$\frac{p}{f_{ck}} = \frac{2.15}{20} = 0.1075$$

For M_{ux1}:

b = 400mm,   D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.151$$

$$\therefore M_{ux1} = 0.151 \times 20 \times 400 \times 500^2 = 302KN.m$$

For M_{uy1}:

b = 500mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.133$$

$$\therefore M_{uy1} = 0.133 \times 20 \times 500 \times 400^2 = 212.8KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.15%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 15.5$$

P_{uz} = 15.5× 400 × 500 = 3100KN

$$\frac{P_u}{P_{uz}} = \frac{1131.58}{3100} = 0.365 > 0.2$$

$\therefore \alpha_n = 1.275$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{285.13}{302}\right)^{1.275} + \left(\frac{28.02}{212.8}\right)^{1.275} = 0.987$$

Column No. 31, 61:

Axial force, P_u = 1174.85KN

Axial stress = $\dfrac{1174.85 \times 10^3}{400 \times 550} = 5.34 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\dfrac{b}{d} = \dfrac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P|}$ = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{550}{30} = 26.43mm$$

or 20mm which ever is greater.

So e_{min} = 26.43mm.

M_{uy} = 1174.85 × 0.02643 = 31.05KN.m

M_{uy} from analysis = 17.32KN.m

So M_{uy} = 31.05KN.m

M_{ux} = 468.22KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1174.85 \times 10^3}{20 \times 400 \times 550} = 0.267$$

Take percentage of steel, p = 2.85%

$$\frac{p}{f_{ck}} = \frac{2.85}{20} = 0.1425$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\dfrac{d'}{D} = \dfrac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.211$$

$$\therefore M_{ux1} = 0.215 \times 20 \times 400 \times 550^2 = 510.62KN.m$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\dfrac{d'}{D} = \dfrac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.184$$

$$\therefore M_{uy1} = 0.176 \times 20 \times 550 \times 400^2 = 323.84 KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.85%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.1$$

$P_{uz} = 19.1 \times 400 \times 550 = 4202$KN

$$\frac{P_u}{P_{uz}} = \frac{1174.85}{4202} = 0.28 > 0.2$$

$\therefore \alpha_n = 1.133$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{468.22}{510.62}\right)^{1.133} + \left(\frac{31.05}{323.84}\right)^{1.133} = 0.989$$

<u>Design of columns of intermediate frame in XZ plane:</u>

Column No. 10, 100:

Axial force, $P_u = 167.14$KN

Axial stress = $\frac{167.14 \times 10^3}{400 \times 500} = 0.836 < 0.1f_{ck} = 2$ N/ mm^2

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50$mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77 mm$$

or 20mm which ever is greater.

So $e_{min} = 22.77$mm.

$M_{uy} = 167.14 \times 0.02277 = 3.8$KN.m

M_{uy} from analysis = 3.53KN.m

So M_{uy} = 3.8KN.m

M_{ux} = 144.41 KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{144.41\times10^3}{20\times400\times500} = 0.042$$

Take percentage of steel, p = 0.85%

$$\frac{p}{f_{ck}} = \frac{0.85}{20} = 0.0425$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.075$$

$$\therefore M_{ux1} = 0.075\times20\times400\times500^2 = 150KN.m$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.069$$

$$\therefore M_{uy1} = 0.069\times20\times500\times400^2 = 110.4KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 0.85%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 11.2$$

P_{uz} = 11.5 × 400 × 500 = 2240KN

$$\frac{P_u}{P_{uz}} = \frac{167.14}{2240} = 0.074 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{144.41}{150}\right)^1 + \left(\frac{3.8}{110.4}\right)^1 = 0.997$$

Column No. 40, 70:

Axial force, P_u = 214.26KN

Axial stress = $\frac{214.26\times10^3}{400\times550}$ = 0.974 < $0.1f_{ck}$ = 2 N/ mm^2

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05\times10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{uy} = 214.26 × 0.02443 = 5.23KN.m

M_{uy} from analysis = 4.39KN.m

So M_{uy} = 5.23KN.m

M_{ux} = 211.24KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{214.26\times10^3}{20\times400\times550} = 0.049$$

Take percentage of steel, p = 1.1%

$$\frac{p}{f_{ck}} = \frac{1.1}{20} = 0.055$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.091$$

$$\therefore M_{ux1} = 0.091\times20\times400\times550^2 = 220.22KN.m$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.083$$

$$\therefore M_{uy1} = 0.083 \times 20 \times 550 \times 400^2 = 146.08 KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.1%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 12.2$$

$P_{uz} = 12.2 \times 400 \times 550 = 2684$KN

$$\frac{P_u}{P_{uz}} = \frac{214.26}{2684} = 0.086 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{21.24}{220.22}\right)^1 + \left(\frac{5.23}{146.08}\right)^1 = 0.992$$

Column No. 9, 99:

Axial force, P_u = 438.55KN

Axial stress = $\frac{438.55 \times 10^3}{400 \times 500} = 2.19 > 0.1 f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 438.55 × 0.02277 = 9.98KN.m

M_{uy} from analysis = 3.55KN.m

So M_{uy} = 9.98KN.m

M_{ux} = 177.23 KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{438.55 \times 10^3}{20 \times 400 \times 500} = 0.11$$

Take percentage of steel, p = 1.0%

$$\frac{p}{f_{ck}} = \frac{1.0}{20} = 0.05$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.097$$

$$\therefore M_{ux1} = 0.097 \times 20 \times 400 \times 500^2 = 194KN.m$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.089$$

$$\therefore M_{uy1} = 0.089 \times 20 \times 500 \times 400^2 = 142.4KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.0%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 12$$

P_{uz} = 12 × 400 × 500 = 2400KN

$$\frac{P_u}{P_{uz}} = \frac{438.55}{2400} = 0.183 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{177.23}{194}\right)^1 + \left(\frac{9.98}{142.4}\right)^1 = 0.984$$

Column No. 39, 69:

Axial force, P_u = 564.89KN

Axial stress = $\frac{564.89 \times 10^3}{400 \times 550} = 2.57$ > $0.1f_{ck}$ = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{uy} = 564.89 × 0.02443 = 13.8KN.m

M_{uy} from analysis = 4.42KN.m

So M_{uy} = 13.8KN.m

M_{ux} = 309.63KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{309.63 \times 10^3}{20 \times 400 \times 550} = 0.128$$

Take percentage of steel, p = 1.7%

$$\frac{p}{f_{ck}} = \frac{1.7}{20} = 0.085$$

For M_{ux1}:

b = 400mm,   D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.137$$

$$\therefore M_{ux1} = 0.137 \times 20 \times 400 \times 550^2 = 331.54KN.m$$

For M_{uy1}:

b = 550mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.121$$

$$\therefore M_{uy1} = 0.121 \times 20 \times 550 \times 400^2 = 212.96KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.7%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 14.2$$

P_{uz} = 14.2 × 400 × 550 = 3124KN

$$\frac{P_u}{P_{uz}} = \frac{564.89}{3124} = 0.181 < 0.2$$

$\therefore \alpha_n = 1$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{309.63}{331.54}\right)^1 + \left(\frac{13.8}{212.96}\right)^1 = 0.994$$

Column No. 8, 98:

Axial force, P_u = 752.45KN

Axial stress = $\frac{752.45 \times 10^3}{400 \times 500} = 3.76 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm   ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50mm$.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 752.45 × 0.02277 = 17.13KN.m

M_{uy} from analysis = 2.75KN.m

So M_{uy} = 17.13KN.m

M_{ux} = 209.31KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{752.45 \times 10^3}{20 \times 400 \times 500} = 0.188$$

Take percentage of steel, p = 1.25%

$$\frac{p}{f_{ck}} = \frac{1.25}{20} = 0.0625$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.112$$

$$\therefore M_{ux1} = 0.112 \times 20 \times 400 \times 500^2 = 224KN.m$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.102$$

$$\therefore M_{uy1} = 0.102 \times 20 \times 500 \times 400^2 = 163.2KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.25%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 12.9$$

$P_{uz} = 12.9 \times 400 \times 500 = 2580KN$

$$\frac{P_u}{P_{uz}} = \frac{752.45}{2580} = 0.292 > 0.2$$

$\therefore \alpha_n = 1.153$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{209.31}{224}\right)^{1.153} + \left(\frac{17.13}{163.2}\right)^{1.153} = 0.999$$

Column No. 38, 68:

Axial force, P_u = 918.01KN

Axial stress = $\frac{918.01 \times 10^3}{400 \times 550} = 4.17 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50mm.$

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

$M_{uy} = 918.01 \times 0.02443 = 22.42KN.m$

M_{uy} from analysis = 3.51KN.m

So M_{uy} = 22.42KN.m

M_{ux} = 383.3KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{918.01 \times 10^3}{20 \times 400 \times 550} = 0.209$$

Take percentage of steel, p = 2.15%

$$\frac{p}{f_{ck}} = \frac{2.15}{20} = 0.1075$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.17$$

$$\therefore M_{ux1} = 0.17 \times 20 \times 400 \times 550^2 = 411.4 KN.m$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.15$$

$$\therefore M_{uy1} = 0.15 \times 20 \times 550 \times 400^2 = 264 KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.15%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 16.3$$

$P_{uz} = 16.3 \times 400 \times 550 = 3586$KN

$$\frac{P_u}{P_{uz}} = \frac{918.01}{3586} = 0.256 > 0.2$$

$\therefore \alpha_n = 1.093$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{383.3}{411.4}\right)^{1.093} + \left(\frac{22.42}{264}\right)^{1.093} = 0.983$$

Column No. 7, 97:

Axial force, P_u = 1083.86KN

Axial stress = $\dfrac{1083.86\times10^3}{400\times500}$ = 5.42 > 0.1f_{ck} = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\dfrac{b}{d}=\dfrac{400}{500}=0.8>0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min}=\frac{3.05\times10^3}{500}+\frac{500}{30}=22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{uy} = 1083.86 × 0.02277 = 24.47KN.m

M_{uy} from analysis = 0.578KN.m

So M_{uy} = 24.47KN.m

M_{ux} = 214.11KN.m

$$\frac{P_u}{f_{ck}bd}=\frac{1083.86\times10^3}{20\times400\times500}=0.271$$

Take percentage of steel, p = 1.4%

$$\frac{p}{f_{ck}}=\frac{1.4}{20}=0.07$$

For M_{ux1}:

b = 400mm, D = 500mm, $\dfrac{d'}{D}=\dfrac{50}{500}=0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2}=0.115$$

$$\therefore M_{ux1}=0.115\times20\times400\times500^2=230KN.m$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.101$$

$$\therefore M_{uy1} = 0.101 \times 20 \times 500 \times 400^2 = 161.6KN.m$$

For $P_{uz:}$

Referring to chart 63 SP 16:1980

Corresponding to p = 1.4%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.2$$

P_{uz} = 13.2× 400 × 500 = 2640KN

$$\frac{P_u}{P_{uz}} = \frac{1083.86}{2640} = 0.446 > 0.2$$

$\therefore \alpha_n = 1.35$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{214.11}{230}\right)^{1.35} + \left(\frac{24.67}{161.6}\right)^{1.35} = 0.987$$

Column No. 37, 97:

Axial force, P_u = 1274.06KN

Axial stress = $\frac{1274.06 \times 10^3}{400 \times 550}$ = 5.79 > 0.1f_{ck} = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{uy} = 1274.06 × 0.02443 = 31.12KN.m

M_{uy} from analysis = 0.394KN.m

So M_{uy} = 31.12KN.m

M_{ux} = 434.98KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1274.06\times 10^3}{20\times 400\times 550} = 0.29$$

Take percentage of steel, p = 2.55%

$$\frac{p}{f_{ck}} = \frac{2.55}{20} = 0.1275$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.19$$

$$\therefore M_{ux1} = 0.19\times 20\times 400\times 550^2 = 459.8KN.m$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.166$$

$$\therefore M_{uy1} = 0.166\times 20\times 550\times 400^2 = 292.16KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.55%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 17.9$$

P_{uz} = 17.9 × 400 × 550 = 3938KN

$$\frac{P_u}{P_{uz}} = \frac{1274.06}{3938} = 0.324 > 0.2$$

$\therefore \alpha_n = 1.207$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{434.98}{459.8}\right)^{1.207} + \left(\frac{31.12}{292.16}\right)^{1.207} = 0.996$$

Column No. 6, 96:

Axial force, P_u = 1323KN

Axial stress = $\frac{1323 \times 10^3}{400 \times 500} = 6.615 > 0.1 f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P|}$ = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{500}{30} = 24.77mm$$

or 20mm which ever is greater.

So e_{min} = 24.77mm.

M_{uy} = 1323 × 0.02477 = 32.77KN.m

M_{uy} from analysis = 0.234KN.m

So M_{uy} = 32.77KN.m

M_{ux} = 282.28KN.m

$$\frac{P_u}{f_{ck} bd} = \frac{1323 \times 10^3}{20 \times 400 \times 500} = 0.331$$

Take percentage of steel, p = 2.3%

$$\frac{p}{f_{ck}} = \frac{2.3}{20} = 0.115$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.15$$

$$\therefore M_{ux1} = 0.151 \times 20 \times 400 \times 500^2 = 300KN.m$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.133$$

$$\therefore M_{uy1} = 0.133 \times 20 \times 500 \times 400^2 = 212.8KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.3%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 15.9$$

P_{uz} = 15.9× 400 × 500 = 3180KN

$$\frac{P_u}{P_{uz}} = \frac{1323}{3180} = 0.416 > 0.2$$

$\therefore \alpha_n = 1.36$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{282.28}{300}\right)^{1.36} + \left(\frac{32.77}{212.8}\right)^{1.36} = 0.995$$

Column No. 36, 66:

Axial force, P_u = 1645.35KN

Axial stress = $\frac{1645.35 \times 10^3}{400 \times 550} = 7.47 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{550}{30} = 26.43mm$$

or 20mm which ever is greater.

So e_{min} = 26.43mm.

M_{uy} = 1645.35 × 0.02643 = 43.49KN.m

M_{uy} from analysis = 0.363KN.m

So M_{uy} = 43.49KN.m

M_{ux} = 464.21KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1645.35 \times 10^3}{20 \times 400 \times 550} = 0.374$$

Take percentage of steel, p = 3.0%

$$\frac{p}{f_{ck}} = \frac{3.0}{20} = 0.15$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.211$$

$$\therefore M_{ux1} = 0.211 \times 20 \times 400 \times 550^2 = 510.62KN.m$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.184$$

$$\therefore M_{uy1} = 0.176 \times 20 \times 550 \times 400^2 = 323.84KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 3.0%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.9$$

P_{uz} = 19.9 × 400 × 550 = 4202KN

$$\frac{P_u}{P_{uz}} = \frac{1645.35}{4378} = 0.374 > 0.2$$

$\therefore \alpha_n = 1.29$

$$\therefore \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{464.21}{510.62}\right)^{1.29} + \left(\frac{43.49}{323.84}\right)^{1.29} = 0.996$$

DESIGN OF FRAME WITH EARTHQUAKE FORCE IN Y DIRECTION:

DESIGN OF BEAMS:

Grade of concrete to be used = M 20

Grade of steel to be used = Fe 415

Assuming 25 Φ bar to be used and 25mm clear cover,

Effective depth d = 450 – 25 – (25/2) = 412.5mm.

According to IS 13920:1993

Width / Depth = (300 / 450) = 0.67> 0.3 ok.

Width = 300mm > 200mm. ok.

Depth < (1/4) × Clear span

$$\frac{1}{4}\left(4000 - \frac{500}{2} - \frac{550}{2}\right) = 868.7mm > 450\text{mm} \quad \text{ok.}$$

$d^{P1} = 25 + (25/2) = 37.5mm$

$$\frac{d'}{d} = \frac{37.5}{412.5} = 0.091$$

From table D of SP 16:1980

$$\frac{M_{u\,lim}}{bd^2} = 2.76 \text{ for M 20 \& Fe 415.}$$

$M_{u\,lim} = 2.76 \times 300 \times (412.5)^2$

$= 140.88$ KN.m

<u>Design of beams of end frame in YZ plane:</u>

Beam No. 211, 231:

<u>For hogging moment:</u>

Maximum hogging moment, $M_u = 306.33$ KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{306.33 \times 10^6}{300 \times 412.5^2} = 6.001$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.93 \\ P_{bottom} = 1.049 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 255.27 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{255.27 \times 10^6}{300 \times 412.5^2} = 5.00$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.645 \\ P_{top} = 0.725 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

$P_{top} = 1.93$

$P_{bottom} = 1.645$

% of reinforcement required at top = $\dfrac{1.93 \times 300 \times 412.5}{100} = 2388.37mm^2$

Provide 2 × 30 Φ + 2 × 25 Φ

A_{st} provided = 2395.44 mm^2.

P_{top} provided = 1.97

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 6.055$$

M_u cal top = 309.09 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.645 \times 300 \times 412.5}{100} = 2035.69 mm^2$

Provide 2 × 30 Φ + 2 × 20 Φ

A_{st} provided = 2042 mm^2.

P_{bottom} provided = 1.65

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.016$$

M_u cal bottom = 256.05 KN.m (Sagging)

Beam No. 212, 232:

For hogging moment:

Maximum hogging moment, M_u = 258.78 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{258.78 \times 10^6}{300 \times 412.5^2} = 5.069$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.666 \\ P_{bottom} &= 0.747 \end{aligned}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 196.85 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{196.85 \times 10^6}{300 \times 412.5^2} = 3.856$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{bottom} &= 1.293 \\ P_{top} &= 0.354 \end{aligned}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.666

P_{bottom} = 1.293

% of reinforcement required at top = $\frac{1.666 \times 300 \times 412.5}{100} = 2061.68mm^2$

Provide 3 × 30 Φ

A_{st} provided = 2120.55 mm^2.

P_{top} provided = 1.714

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.223$$

M_u cal top = 266.62 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.293 \times 300 \times 412.5}{100} = 1600.08mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{bottom} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal bottom = 198.22 KN.m (Sagging)

Beam No. 213, 233:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 226.64 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{226.64 \times 10^6}{300 \times 412.5^2} = 4.44$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.472 \\ P_{bottom} = 0.543 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 162.38 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{162.38 \times 10^6}{300 \times 412.5^2} = 3.181$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.085 \\ P_{bottom} = 0.136 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.472

P_{bottom} = 1.085

% of reinforcement required at top = $\dfrac{1.472 \times 300 \times 412.5}{100} = 1821.6mm^2$

Provide 2 × 30 Φ + 1 × 25 Φ

A_{st} provided = 1904.57 mm^2.

P_{top} provided = 1.539

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.654$$

M_u cal top = 237.57 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.085 \times 300 \times 412.5}{100} = 1342.68mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{bottom} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal bottom = 167.79 KN.m (Sagging)

Beam No. 214, 234:

For hogging moment:

Maximum hogging moment, M_u = 161.32 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{161.32 \times 10^6}{300 \times 412.5^2} = 3.16$$

Referring to table 50 of SP 16:1980

P_{top} = 1.0786

P_{bottom} = 0.129 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 96.01 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{96.01 \times 10^6}{300 \times 412.5^2} = 1.88$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.595 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.0786

P_{bottom} = 0.595

% of reinforcement required at top = $\frac{1.0786 \times 300 \times 412.5}{100} = 1334.77 mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{top} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal top = 167.79 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.595 \times 300 \times 412.5}{100} = 736.31 mm^2$

Provide 4 × 16 Φ

A_{st} provided = 804.24 mm^2.

P_{bottom} provided = 0.649

Referring to table 2 of SP 16:1980

$\dfrac{M_u}{bd^2} = 2.025$

M_u cal bottom = 103.37 KN.m (Sagging)

Beam No. 215, 235:

For hogging moment:

Maximum hogging moment, M_u = 67.8 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{67.8 \times 10^6}{300 \times 412.5^2} = 1.328$$

Referring to table 2 of SP 16:1980

P_{top} = 0.402 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 34.5 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{34.5 \times 10^6}{300 \times 412.5^2} = 0.676$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.195 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.402

P_{bottom} = 0.195

% of reinforcement required at top = $\dfrac{0.402 \times 300 \times 412.5}{100} = 497.48 mm^2$

Provide 2 × 16 Φ + 1 × 12 Φ

A_{st} provided = 515.21 mm^2.

P_{top} provided = 0.416

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.371$$

M_u cal top = 69.99 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.195 \times 300 \times 412.5}{100} = 241.31 mm^2$

Provide 2 × 12 Φ + 1 × 10 Φ

A_{st} provided = 304.71 mm^2.

P_{bottom} provided = 0.246

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.843$$

M_u cal bottom = 43.03 KN.m (Sagging)

Beam No. 216, 226:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 248.67 KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{248.67 \times 10^6}{300 \times 412.5^2} = 4.871$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.605 \\ P_{bottom} = 0.683 \end{array}\right] \text{---------} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 182.92 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{182.92 \times 10^6}{300 \times 412.5^2} = 3.583$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.209 \\ P_{top} = 0.266 \end{array}\right] \text{---------} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.605

P_{bottom} = 1.209

% of reinforcement required at top = $\dfrac{1.605 \times 300 \times 412.5}{100} = 1986.19 mm^2$

Provide 2 × 30 Φ + 2 × 20 Φ

A_{st} provided = 2042 mm^2.

P_{top} provided = 1.65

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.016$$

M_u cal top = 256.05 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.209 \times 300 \times 412.5}{100} = 1496.14 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{bottom} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal bottom = 198.22 KN.m (Sagging)

Beam No. 217, 227:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 228.04 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{228.04 \times 10^6}{300 \times 412.5^2} = 4.467$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.481 \\ P_{bottom} = 0.552 \end{array}\right\} \text{— — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 162.07 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{162.07 \times 10^6}{300 \times 412.5^2} = 3.175$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.083 \\ P_{top} & = 0.134 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.481

P_{bottom} = 1.083

% of reinforcement required at top = $\frac{1.481 \times 300 \times 412.5}{100} = 1832.73mm^2$

Provide 2 × 30 Φ + 1 × 25 Φ

A_{st} provided = 1904.57 mm^2.

P_{top} provided = 1.539

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.654$$

M_u cal top = 237.57 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.083 \times 300 \times 412.5}{100} = 1340.21mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{bottom} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal bottom = 167.79 KN.m (Sagging)

Beam No. 218, 228:

<u>For hogging moment:</u>

Maximum hogging moment, $M_u = 197.86$ KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{197.86 \times 10^6}{300 \times 412.5^2} = 3.876$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} \quad = 1.299 \\ P_{bottom} = 0.361 \end{array}\right\} \text{— — — — —} \ \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 131.64 KN.m < $M_{u\,lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{131.64 \times 10^6}{300 \times 412.5^2} = 2.579$$

Referring to table 2 of SP 16:1980

$P_{bottom} = 0.874$ — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

$P_{top} = 1.299$

$P_{bottom} = 0.874$

% of reinforcement required at top = $\dfrac{1.299 \times 300 \times 412.5}{100} = 1607.51 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{top} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal top = 198.22 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.874 \times 300 \times 412.5}{100} = 1081.58 mm^2$

Provide 2 × 25 Φ + 1 × 12 Φ

A_{st} provided = 1094.83 mm^2.

P_{bottom} provided = 0.8847

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.604$$

M_u cal bottom = 132.92 KN.m (Sagging)

Beam No. 219, 229:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 149.2 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{149.2 \times 10^6}{300 \times 412.5^2} = 2.923$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.005 \\ P_{bottom} = 0.052 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 84.08 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{84.08 \times 10^6}{300 \times 412.5^2} = 1.647$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.511 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.005

P_{bottom} = 0.551

% of reinforcement required at top = $\dfrac{1.005 \times 300 \times 412.5}{100} = 1243.69 mm^2$

Provide 4 × 20 Φ

A_{st} provided = 1256.6 mm^2.

P_{top} provided = 1.015

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.955$$

M_u cal top = 150.84 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.551 \times 300 \times 412.5}{100} = 681.86 mm^2$

Provide 2 × 20 Φ + 1 × 10 Φ

A_{st} provided = 706.83 mm^2.

P_{bottom} provided = 0.571

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.816$$

M_u cal bottom = 92.7 KN.m (Sagging)

Beam No. 220, 230:

For hogging moment:

Maximum hogging moment, M_u = 66.87 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{66.87 \times 10^6}{300 \times 412.5^2} = 1.31$$

Referring to table 2 of SP 16:1980

P_{top} = 0.3954 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 32.88 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{32.88 \times 10^6}{300 \times 412.5^2} = 0.644$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.1852 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.3954

P_{bottom} = 0.1852

% of reinforcement required at top = $\dfrac{0.3954 \times 300 \times 412.5}{100} = 489.3 mm^2$

Provide $2 \times 16\ \Phi + 1 \times 12\ \Phi$

A_{st} provided = 515.21 mm^2.

P_{top} provided = 0.416

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.371$$

M_u cal top = 69.99 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.1852 \times 300 \times 412.5}{100} = 229.8 mm^2$

Provide $3 \times 10\ \Phi$

A_{st} provided = 235.59 mm^2.

P_{bottom} provided = 0.19

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.659$$

M_u cal bottom = 33.64 KN.m (Sagging)

Beam No.221:

For hogging moment:

Maximum hogging moment, M_u = 252.24 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{252.24 \times 10^6}{300 \times 412.5^2} = 4.941$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.627 \\ P_{bottom} = 0.706 \end{array}\right\} \text{-----} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 186.29 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{186.29 \times 10^6}{300 \times 412.5^2} = 3.649$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.229 \\ P_{tpo} & = 0.287 \end{array}\right] \text{--- --- --- ---} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.627

P_{bottom} = 1.229

% of reinforcement required at top = $\dfrac{1.627 \times 300 \times 412.5}{100} = 2013.42mm^2$

Provide 2 × 30 Φ + 2 × 20 Φ

A_{st} provided = 2042 mm^2.

P_{top} provided = 1.65

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.016$$

M_u cal top = 256.05 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.227 \times 300 \times 412.5}{100} = 1520.89mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{bottom} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal bottom = 198.22 KN.m (Sagging)

Beam No. 222:

For hogging moment:

Maximum hogging moment, M_u = 226.45 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{226.45 \times 10^6}{300 \times 412.5^2} = 4.436$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{top} & = 1.471 \\ & \end{array}\right] \text{--- --- --- ---} \boxed{1}$$

P_{bottom} = 0.542

<u>For sagging moment:</u>

Maximum sagging moment = 160.31 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{160.31 \times 10^6}{300 \times 412.5^2} = 3.14$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{ll} P_{bottom} & = 1.072 \\ P_{top} & = 0.122 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.471

P_{bottom} = 1.072

% of reinforcement required at top = $\dfrac{1.471 \times 300 \times 412.5}{100} = 1820.36 mm^2$

Provide 2 × 30 Φ + 1 × 25 Φ

A_{st} provided = 1904.57 mm^2.

P_{top} provided = 1.539

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.654$$

M_u cal top = 237.57 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.072 \times 300 \times 412.5}{100} = 1326.6 mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{bottom} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal bottom = 167.79 KN.m (Sagging)

Beam No.223:

For hogging moment:

Maximum hogging moment, $M_u = 196.52$ KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{196.52 \times 10^6}{300 \times 412.5^2} = 3.85$$

Referring to table 50 of SP 16:1980

P_{top} = 1.291

P_{bottom} = 0.353 $\boxed{1}$

For sagging moment:

Maximum sagging moment = 130.37 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{130.37 \times 10^6}{300 \times 412.5^2} = 2.579$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.8633 $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.291

P_{bottom} = 0.8633

% of reinforcement required at top = $\frac{1.291 \times 300 \times 412.5}{100} = 1597.61 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{top} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal top = 198.22 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.8633 \times 300 \times 412.5}{100} = 1068.33 mm^2$

Provide 2 × 25 Φ + 1 × 12 Φ

A_{st} provided = 1094.83 mm^2.

P_{bottom} provided = 0.8847

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.604$$

M_u cal bottom = 132.92 KN.m (Sagging)

Beam No.224:

For hogging moment:

Maximum hogging moment, M_u = 146.61 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{146.61 \times 10^6}{300 \times 412.5^2} = 2.872$$

Referring to table 50 of SP 16:1980

P_{top} = 0.9896

P_{bottom} = 0.036 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 80.54 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{80.54 \times 10^6}{300 \times 412.5^2} = 1.578$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.487 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.9896

P_{bottom} = 0.487

% of reinforcement required at top = $\dfrac{0.9896 \times 300 \times 412.5}{100} = 1224.63mm^2$

Provide 4 × 20 Φ

A_{st} provided = 1256.6 mm^2.

P_{top} provided = 1.015

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.955$$

M_u cal top = 150.84 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.487 \times 300 \times 412.5}{100} = 602.66mm^2$

Provide 3 × 16 Φ

A_{st} provided = 603.18 mm^2.

P_{bottom} provided = 0.4874

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.581$$

M_u cal bottom = 80.71 KN.m (Sagging)

Beam No. 225:

For hogging moment:

Maximum hogging moment, M_u = 63.73 KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{63.73 \times 10^6}{300 \times 412.5^2} = 1.248$$

Referring to table 2 of SP 16:1980

P_{top} = 0.3753 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 27.47 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{27.47 \times 10^6}{300 \times 412.5^2} = 0.538$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.1544 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.3753

P_{bottom} = 0.1544

% of reinforcement required at top = $\dfrac{0.3753 \times 300 \times 412.5}{100} = 464.43mm^2$

Provide 2 × 16 Φ + 1 × 10 Φ

A_{st} provided = 480.65 mm^2.

P_{top} provided = 0.3884

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.289$$

M_u cal top = 65.8 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.1544 \times 300 \times 412.5}{100} = 191.07 mm^2$

Provide 2 × 12 Φ

A_{st} provided = 226.18 mm^2.

P_{bottom} provided = 0.1828

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.636$$

M_u cal bottom = 32.47 KN.m (Sagging)

<u>Design of columns of end frame in YZ plane:</u>

Column No. 1, 26:

Axial force, P_u = 1135.77KN

Axial stress = $\frac{1135.77 \times 10^3}{400 \times 500} = 5.679 > 0.1 f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05\times10^3}{500} + \frac{500}{30} = 24.77mm$$

or 20mm which ever is greater.

So e_{min} = 24.77mm.

M_{ux} = 1135.77 × 0.02477 = 28.13KN.m

M_{ux} from analysis = 41.26KN.m

So M_{ux} = 41.26KN.m

M_{uy} = 268.74KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1135.77\times10^3}{20\times400\times500} = 0.284$$

Take percentage of steel, p = 2.65%

$$\frac{p}{f_{ck}} = \frac{2.65}{20} = 0.1325$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.18$$

$$\therefore M_{uy1} = 0.18\times20\times500\times400^2 = 288KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.202$$

$$\therefore M_{ux1} = 0.202\times20\times400\times500^2 = 404KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.65%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 18.6$$

P_{uz} = 18.6× 400 × 500 = 3720KN

$$\frac{P_u}{P_{uz}} = \frac{1135.77}{3720} = 0.305 > 0.2$$

$\therefore \alpha_n = 1.175$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{268.74}{288}\right)^{1.175} + \left(\frac{41.26}{404}\right)^{1.175} = 0.99$$

Column No. 6, 21:

Axial force, P_u = 1199.79KN

Axial stress = $\frac{1199.79 \times 10^3}{400 \times 500} = 5.999 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P!}$ = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{min} = \frac{4.05 \times 10^3}{500} + \frac{500}{30} = 24.77mm$$

or 20mm which ever is greater.

So e_{min} = 24.77mm.

M_{ux} = 1199.79 × 0.02477 = 29.72KN.m

M_{ux} from analysis = 43.51KN.m

So M_{ux} = 43.51KN.m

M_{uy} = 385.63KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1199.79 \times 10^3}{20 \times 400 \times 500} = 0.299$$

Take percentage of steel, p = 3.0%

$$\frac{p}{f_{ck}} = \frac{3.0}{20} = 0.15$$

For M_{uy1}:

b = 500mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.254$$

$$\therefore M_{uy1} = 0.254 \times 20 \times 500 \times 400^2 = 406.4KN.m$$

<u>For M_{ux1}:</u>

b = 400mm,   D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.268$$

$$\therefore M_{ux1} = 0.268 \times 20 \times 400 \times 500^2 = 536KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 3.0%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 20.2$$

$P_{uz} = 20.2 \times 400 \times 500 = 4040$KN

$$\frac{P_u}{P_{uz}} = \frac{1199.79}{4040} = 0.297 > 0.2$$

$\therefore \alpha_n = 1.16$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{385.63}{406.4}\right)^{1.16} + \left(\frac{43.51}{536}\right)^{1.16} = 0.996$$

Column No. 11, 16:

Axial force, P_u = 1207.59KN

Axial stress = $\frac{1207.59 \times 10^3}{400 \times 500} = 6.03 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm   ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50$mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{500}{30} = 24.77mm$$

or 20mm which ever is greater.

So e_{min} = 24.77mm.

M_{ux} = 1207.59 × 0.02477 = 29.91KN.m

M_{ux} from analysis = 43.73KN.m

So M_{ux} = 43.73KN.m

M_{uy} = 347.33KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1207.59 \times 10^3}{20 \times 400 \times 500} = 0.302$$

Take percentage of steel, p = 2.8%

$$\frac{p}{f_{ck}} = \frac{2.8}{20} = 0.14$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.232$$

$$\therefore M_{uy1} = 0.232 \times 20 \times 500 \times 400^2 = 371.2KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.248$$

$$\therefore M_{ux1} = 0.248 \times 20 \times 400 \times 500^2 = 496KN.m$$

For $P_{uz:}$

Referring to chart 63 SP 16:1980

Corresponding to p = 2.8%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.8$$

P_{uz} = 19.8× 400 × 500 = 3960KN

$$\frac{P_u}{P_{uz}} = \frac{1207.59}{3960} = 0.305 > 0.2$$

$\therefore \alpha_n = 1.175$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{347.33}{371.2}\right)^{1.175} + \left(\frac{43.73}{496}\right)^{1.175} = 0.99$$

Column No. 2, 27:

Axial force, P_u = 844.95KN

Axial stress = $\frac{844.95 \times 10^3}{400 \times 500} = 4.22 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 844.95 × 0.02277 = 19.24KN.m

M_{ux} from analysis = 43.22KN.m

So M_{ux} = 43.22KN.m

M_{uy} = 189.77KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{844.95 \times 10^3}{20 \times 400 \times 500} = 0.211$$

Take percentage of steel, p = 1.825%

$$\frac{p}{f_{ck}} = \frac{1.825}{20} = 0.09125$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.133$$

$$\therefore M_{uy1} = 0.133 \times 20 \times 500 \times 400^2 = 212.2 KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.149$$

$$\therefore M_{ux1} = 0.149 \times 20 \times 400 \times 500^2 = 298 KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.825%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 14.9$$

P_{uz} = 14.9× 400 × 500 = 2980KN

$$\frac{P_u}{P_{uz}} = \frac{844.59}{2980} = 0.284 > 0.2$$

$\therefore \alpha_n = 1.14$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{189.77}{212.8}\right)^{1.14} + \left(\frac{43.22}{298}\right)^{1.14} = 0.988$$

Column No. 7, 22:

Axial force, P_u = 937.65KN

Axial stress = $\frac{937.65 \times 10^3}{400 \times 500}$ = 4.688 > $0.1f_{ck}$ = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P!}$ = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 937.65 × 0.02277 = 21.35KN.m

M_{ux} from analysis = 48.89KN.m

So M_{ux} = 48.89KN.m

M_{uy} = 322.72KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{937.65 \times 10^3}{20 \times 400 \times 500} = 0.234$$

Take percentage of steel, p = 2.6%

$$\frac{p}{f_{ck}} = \frac{2.6}{20} = 0.13$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.223$$

$$\therefore M_{uy1} = 0.223 \times 20 \times 500 \times 400^2 = 356.8KN.m$$

For M_{ux1}:

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.243$$

$$\therefore M_{ux1} = 0.243 \times 20 \times 400 \times 500^2 = 486KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.6%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.4$$

P_{uz} = 19.4× 400 × 500 = 3880KN

$$\frac{P_u}{P_{uz}} = \frac{937.65}{3880} = 0.241 > 0.2$$

$\therefore \alpha_n = 1.068$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{322.72}{356.8}\right)^{1.068} + \left(\frac{48.89}{486}\right)^{1.068} = 0.992$$

Column No. 12, 17:

Axial force, P_u = 948.65KN

Axial stress = $\frac{948.65 \times 10^3}{400 \times 500} = 4.743 > 0.1 f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 948.65 × 0.02277 = 21.6KN.m

M_{ux} from analysis = 48.94KN.m

So M_{ux} = 48.94KN.m

M_{uy} = 298.69KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{948.65 \times 10^3}{20 \times 400 \times 500} = 0.237$$

Take percentage of steel, p = 2.35%

$$\frac{p}{f_{ck}} = \frac{2.35}{20} = 0.1175$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.206$$

$$\therefore M_{uy1} = 0.206 \times 20 \times 500 \times 400^2 = 329.6KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.235$$

$$\therefore M_{ux1} = 0.235 \times 20 \times 400 \times 500^2 = 470KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.35%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.1$$

P_{uz} = 19.1× 400 × 500 = 3820KN

$$\frac{P_u}{P_{uz}} = \frac{948.65}{3820} = 0.248 > 0.2$$

$\therefore \alpha_n = 1.08$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{298.69}{329.6}\right)^{1.08} + \left(\frac{48.94}{470}\right)^{1.08} = 0.992$$

Column No. 3, 28:

Axial force, P_u = 579.72KN

Axial stress = $\frac{579.72 \times 10^3}{400 \times 500}$ = 2.899 > 0.1f_{ck} = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P|}$ = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 579.72 × 0.02277 = 13.2KN.m

M_{ux} from analysis = 42.22KN.m

So M_{ux} = 42.22KN.m

M_{uy} = 192.82KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{579.72 \times 10^3}{20 \times 400 \times 500} = 0.145$$

Take percentage of steel, p = 1.6%

$$\frac{p}{f_{ck}} = \frac{1.6}{20} = 0.08$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.141$$

$$\therefore M_{uy1} = 0.141 \times 20 \times 500 \times 400^2 = 225.6 KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.157$$

$$\therefore M_{ux1} = 0.157 \times 20 \times 400 \times 500^2 = 314 KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.6%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 15.2$$

P_{uz} = 15.2× 400 × 500 = 3040KN

$$\frac{P_u}{P_{uz}} = \frac{579.72}{3040} = 0.190 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{192.92}{225.6}\right)^{1.0} + \left(\frac{42.22}{314}\right)^{1.0} = 0.99$$

Column No. 8, 23:

Axial force, P_u = 681.15KN

Axial stress = $\frac{681.15 \times 10^3}{400 \times 500} = 3.4 > 0.1 f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{P1} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 681.15 × 0.02277 = 15.5KN.m

M_{ux} from analysis = 47.23KN.m

So M_{ux} = 47.23KN.m

M_{uy} = 288.02KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{681.15 \times 10^3}{20 \times 400 \times 500} = 0.17$$

Take percentage of steel, p = 2.15%

$$\frac{p}{f_{ck}} = \frac{2.15}{20} = 0.1075$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.204$$

$$\therefore M_{uy1} = 0.204 \times 20 \times 500 \times 400^2 = 326.4KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.219$$

$$\therefore M_{ux1} = 0.219 \times 20 \times 400 \times 500^2 = 438KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.15%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 18.8$$

P_{uz} = 18.8× 400 × 500 = 3760KN

$$\frac{P_u}{P_{uz}} = \frac{681.15}{3760} = 0.181 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{288.02}{326.4}\right)^{1.0} + \left(\frac{47.23}{438}\right)^{1.0} = 0.99$$

Column No. 13, 18:

Axial force, P_u = 690.59KN

Axial stress = $\frac{690.59 \times 10^3}{400 \times 500} = 3.45 > 0.1 f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 690.59 × 0.02277 = 15.72KN.m

M_{ux} from analysis = 47.29KN.m

So M_{ux} = 47.29KN.m

M_{uy} = 268.83KN.m

$$\frac{P_u}{f_{ck} bd} = \frac{690.59 \times 10^3}{20 \times 400 \times 500} = 0.173$$

Take percentage of steel, p = 2.0%

$$\frac{p}{f_{ck}} = \frac{2.0}{20} = 0.10$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.189$$

$$\therefore M_{uy1} = 0.189 \times 20 \times 500 \times 400^2 = 302.4KN.m$$

For M_{ux1}:

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.213$$

$$\therefore M_{ux1} = 0.213 \times 20 \times 400 \times 500^2 = 426KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.0%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 18.4$$

$P_{uz} = 18.4 \times 400 \times 500 = 3680$KN

$$\frac{P_u}{P_{uz}} = \frac{690.59}{3680} = 0.188 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{268.63}{302.4}\right)^{1.0} + \left(\frac{47.29}{426}\right)^{1.0} = 0.999$$

Column No. 4, 29:

Axial force, $P_u = 302.4$KN

Axial stress = $\frac{302.4 \times 10^3}{400 \times 500} = 1.512 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 302.4 × 0.02277 = 6.88KN.m

M_{ux} from analysis = 35.4KN.m

So M_{ux} = 35.4KN.m

M_{uy} = 140.06KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{302.4 \times 10^3}{20 \times 400 \times 500} = 0.0756$$

Take percentage of steel, p = 1.4%

$$\frac{p}{f_{ck}} = \frac{1.4}{20} = 0.07$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.104$$

$$\therefore M_{uy1} = 0.104 \times 20 \times 500 \times 400^2 = 166.4KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.114$$

$$\therefore M_{ux1} = 0.114 \times 20 \times 400 \times 500^2 = 228KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.4%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.2$$

P_{uz} = 13.2× 400 × 500 = 2640KN

$$\frac{P_u}{P_{uz}} = \frac{302.4}{2640} = 0.12 < 0.2$$

∴ α_n = 1.0

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{140.06}{164.4}\right)^{1.0} + \left(\frac{35.4}{228}\right)^{1.0} = 0.993$$

Column No. 9, 24:

Axial force, P_u = 424.32KN

Axial stress = $\frac{424.32\times10^3}{400\times500}$ = 2.12 > 0.1f_{ck} = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{500} = 0.8 > 0.4 \qquad \text{ok.}$$

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05\times10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 424.32 × 0.02277 = 9.66KN.m

M_{ux} from analysis = 45.83KN.m

So M_{ux} = 45.83KN.m

M_{uy} = 233.88KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{424.32\times10^3}{20\times400\times500} = 0.106$$

Take percentage of steel, p = 1.675%

$$\frac{p}{f_{ck}} = \frac{1.675}{20} = 0.08375$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.167$$

$$\therefore M_{uy1} = 0.167 \times 20 \times 500 \times 400^2 = 267.2KN.m$$

For M_{ux1}:

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.189$$

$$\therefore M_{ux1} = 0.189 \times 20 \times 400 \times 500^2 = 378KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.675%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 17.4$$

P_{uz} = 17.4× 400 × 500 = 3480KN

$$\frac{P_u}{P_{uz}} = \frac{424.32}{3480} = 0.122 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{233.88}{267.2}\right)^{1.0} + \left(\frac{45.83}{378}\right)^{1.0} = 0.997$$

Column No. 14, 19:

Axial force, P_u = 431.05KN

Axial stress = $\dfrac{431.05\times10^3}{400\times500} = 2.155 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\dfrac{b}{d} = \dfrac{400}{500} = 0.8 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50$mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05\times10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 431.05 × 0.02277 = 9.815KN.m

M_{ux} from analysis = 45.88KN.m

So M_{ux} = 45.88KN.m

M_{uy} = 224.07KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{431.05\times10^3}{20\times400\times500} = 0.108$$

Take percentage of steel, p = 1.525%

$$\frac{p}{f_{ck}} = \frac{1.525}{20} = 0.07625$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\dfrac{d'}{D} = \dfrac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.161$$

$$\therefore M_{uy1} = 0.161\times20\times500\times400^2 = 257.6KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\dfrac{d'}{D} = \dfrac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.183$$

$$\therefore M_{ux1} = 0.183 \times 20 \times 400 \times 500^2 = 366KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.525%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 17.1$$

P_{uz} = 17.1× 400 × 500 = 3420KN

$$\frac{P_u}{P_{uz}} = \frac{431.05}{3420} = 0.126 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{224.07}{257.6}\right)^{1.0} + \left(\frac{45.88}{372}\right)^{1.0} = 0.995$$

Column No. 5, 30:

Axial force, P_u = 119.18KN

Axial stress = $\frac{119.18 \times 10^3}{400 \times 500} = 0.598 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 119.18 × 0.02277 = 2.72KN.m

M_{ux} from analysis = 38.42KN.m

So M_{ux} = 38.42KN.m

M_{uy} = 97.99KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{119.18 \times 10^3}{20 \times 400 \times 500} = 0.0298$$

Take percentage of steel, p = 1.15%

$$\frac{p}{f_{ck}} = \frac{1.15}{20} = 0.0575$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.08$$

$$\therefore M_{uy1} = 0.08 \times 20 \times 500 \times 400^2 = 128KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.089$$

$$\therefore M_{ux1} = 0.089 \times 20 \times 400 \times 500^2 = 178KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.15%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 12.5$$

P_{uz} = 12.5× 400 × 500 = 2500KN

$$\frac{P_u}{P_{uz}} = \frac{119.18}{2500} = 0.047 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{97.99}{128}\right)^{1.0} + \left(\frac{38.42}{178}\right)^{1.0} = 0.981$$

Column No. 10, 25:

Axial force, P_u = 167.14KN

Axial stress = $\frac{167.14\times10^3}{400\times500} = 0.8359 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{\min} = \frac{3.05\times10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 167.14 × 0.02277 = 3.807KN.m

M_{ux} from analysis = 48KN.m

So M_{ux} = 48KN.m

M_{uy} = 158.26KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{167.14\times10^3}{20\times400\times500} = 0.042$$

Take percentage of steel, p = 1.45%

$$\frac{p}{f_{ck}} = \frac{1.45}{20} = 0.0725$$

<u>For M_{uy1}:</u>

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.122$$

$$\therefore M_{uy1} = 0.122\times20\times500\times400^2 = 195.2KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.137$$

$$\therefore M_{ux1} = 0.137 \times 20 \times 400 \times 500^2 = 274 KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.45%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 14.7$$

P_{uz} = 14.7× 400 × 500 = 2940KN

$$\frac{P_u}{P_{uz}} = \frac{167.14}{2940} = 0.057 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{158.26}{195.2}\right)^{1.0} + \left(\frac{48}{274}\right)^{1.0} = 0.986$$

Column No. 15, 20:

Axial force, P_u = 169.83KN

Axial stress = $\frac{169.83 \times 10^3}{400 \times 500} = 0.849 < 0.1 f_{ck}$ = 2 N/ mm^2

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 3.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{500}{30} = 22.77mm$$

or 20mm which ever is greater.

So e_{min} = 22.77mm.

M_{ux} = 169.83 × 0.02277 = 3.867KN.m

M_{ux} from analysis = 48.11KN.m

So M_{ux} = 48.11KN.m

M_{uy} = 143.46KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{169.83 \times 10^3}{20 \times 400 \times 500} = 0.042$$

Take percentage of steel, p = 1.3%

$$\frac{p}{f_{ck}} = \frac{1.3}{20} = 0.065$$

For M_{uy1}:

b = 500mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.112$$

$$\therefore M_{uy1} = 0.112 \times 20 \times 500 \times 400^2 = 179.2 KN.m$$

For M_{ux1}:

b = 400mm, D = 500mm, $\frac{d'}{D} = \frac{50}{500} = 0.1$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.126$$

$$\therefore M_{ux1} = 0.126 \times 20 \times 400 \times 500^2 = 252 KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.3%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 14.2$$

$P_{uz} = 14.2 \times 400 \times 500 = 2840$KN

$$\frac{P_u}{P_{uz}} = \frac{169.83}{2840} = 0.06 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{143.46}{179.2}\right)^{1.0} + \left(\frac{48.11}{252}\right)^{1.0} = 0.991$$

Design of beams of intermediate frame in YZ plane:

Beam No. 236, 256:

For hogging moment:

Maximum hogging moment, $M_u = 322.38$ KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{322.38 \times 10^6}{300 \times 412.5^2} = 6.315$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 2.05 \\ P_{bottom} = 1.15 \end{array}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 261.18 KN.m > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{261.18 \times 10^6}{300 \times 412.5^2} = 5.116$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.681 \\ P_{top} = 0.762 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.05

P_{bottom} = 1.681

% of reinforcement required at top = $\dfrac{2.05 \times 300 \times 412.5}{100} = 2536.88 mm^2$

Provide 2 × 32 Φ + 2 × 25 Φ

A_{st} provided = 2590.22 mm^2.

P_{top} provided = 2.093

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 6.455$$

M_u cal top = 329.5 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.681 \times 300 \times 412.5}{100} = 2080 mm^2$

Provide 3 × 30 Φ

A_{st} provided = 2120.55 mm^2.

P_{bottom} provided = 1.714

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.223$$

M_u cal bottom = 266.62 KN.m (Sagging)

Beam No. 237, 257:

For hogging moment:

Maximum hogging moment, M_u = 273.37 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{273.37 \times 10^6}{300 \times 412.5^2} = 5.355$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.754 \\ P_{bottom} = 0.84 \end{array}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 200.04 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{200.04 \times 10^6}{300 \times 412.5^2} = 3.92$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.312 \\ P_{top} = 0.375 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.754

P_{bottom} = 1.312

% of reinforcement required at top = $\dfrac{1.754 \times 300 \times 412.5}{100} = 2170.58 mm^2$

Provide 2 × 32 Φ + 2 × 20 Φ

A_{st} provided = 2236.76 mm^2.

P_{top} provided = 1.807

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.526$$

M_u cal top = 282.09KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.312 \times 300 \times 412.5}{100} = 1623.6mm^2$

Provide 2 × 30 Φ + 2 × 12 Φ

A_{st} provided = 1639.88 mm^2.

P_{bottom} provided = 1.325

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.961$$

M_u cal bottom = 202.2 KN.m (Sagging)

Beam No. 238, 258:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 239.64 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{239.64 \times 10^6}{300 \times 412.5^2} = 4.695$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.551 \\ P_{bottom} = 0.626 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 163.59 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{163.59 \times 10^6}{300 \times 412.5^2} = 3.205$$

Referring to table 50 of SP 16:1980

P_{bottom} = 1.093

P_{top} = 0.111 $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.551

P_{bottom} = 1.093

% of reinforcement required at top = $\frac{1.551 \times 300 \times 412.5}{100} = 1912.36 mm^2$

Provide 2 × 32 Φ + 2 × 16 Φ

A_{st} provided = 2010.66 mm^2.

P_{top} provided = 1.625

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.93$$

M_u cal top = 251.66 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.093 \times 300 \times 412.5}{100} = 1352.58 mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{bottom} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal bottom = 167.79 KN.m (Sagging)

Beam No. 239, 259:

For hogging moment:

Maximum hogging moment, M_u = 172.02 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{172.02 \times 10^6}{300 \times 412.5^2} = 3.37$$

Referring to table 50 of SP 16:1980

P_{top} = 1.143

P_{bottom} = 0.197 — — — — — $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 95.01 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{95.01 \times 10^6}{300 \times 412.5^2} = 1.869$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.591 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.143

P_{bottom} = 0.591

% of reinforcement required at top = $\dfrac{1.143 \times 300 \times 412.5}{100} = 1414.46 mm^2$

Provide 3 × 25 Φ

A_{st} provided = 1472.61 mm^2.

P_{top} provided = 1.19

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.522$$

M_u cal top = 179.79 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.591 \times 300 \times 412.5}{100} = 731.36 mm^2$

Provide 4 × 16 Φ

A_{st} provided = 804.24 mm^2.

P_{bottom} provided = 0.649

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.025$$

M_u cal bottom = 103.37 KN.m (Sagging)

Beam No. 240, 260:

For hogging moment:

Maximum hogging moment, $M_u = 71.37$ KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{71.37 \times 10^6}{300 \times 412.5^2} = 1.398$$

Referring to table 2 of SP 16:1980

P_{top} = 0.425 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 35.68 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{35.68 \times 10^6}{300 \times 412.5^2} = 0.699$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.2027 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.425

P_{bottom} = 0.2027

% of reinforcement required at top = $\frac{0.425 \times 300 \times 412.5}{100} = 525.93 mm^2$

Provide 2 × 16 Φ + 2 × 10 Φ

A_{st} provided = 559.18 mm^2.

P_{top} provided = 0.452

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.476$$

M_u cal top = 75.34 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.2027 \times 300 \times 412.5}{100} = 250.84 mm^2$

Provide 2 × 12 Φ + 1 × 10 Φ

A_{st} provided = 304.71 mm^2.

P_{bottom} provided = 0.246

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.843$$

M_u cal bottom = 43.03 KN.m (Sagging)

Beam No. 241, 251:

For hogging moment:

Maximum hogging moment, M_u = 266.04 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{266.04 \times 10^6}{300 \times 412.5^2} = 5.212$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.7106 \\ P_{bottom} &= 0.794 \end{aligned}\right\} \text{— — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 189.06 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{189.06 \times 10^6}{300 \times 412.5^2} = 3.704$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{bottom} &= 1.247 \\ P_{top} &= 0.306 \end{aligned}\right\} \text{— — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.7106

P_{bottom} = 1.247

% of reinforcement required at top = $\frac{1.7106 \times 300 \times 412.5}{100} = 2116.86 mm^2$

Provide 3 × 30 Φ

A_{st} provided = 2120.55 mm^2.

P_{top} provided = 1.714

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.223$$

M_u cal top = 266.62 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.247 \times 300 \times 412.5}{100} = 1543.16 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{bottom} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal bottom = 198.22 KN.m (Sagging)

Beam No. 242, 252:

For hogging moment:

Maximum hogging moment, M_u = 243.48 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{243.48 \times 10^6}{300 \times 412.5^2} = 4.77$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{top} &= 1.574 \\ P_{bottom} &= 0.65 \end{aligned}\right\} \text{— — — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 166.4 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{166.04 \times 10^6}{300 \times 412.5^2} = 3.26$$

Referring to table 50 of SP 16:1980

$$\left.\begin{aligned} P_{b0tt0m} &= 1.1096 \\ P_{bottom} &= 0.161 \end{aligned}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.574

P_{top} = 1.1096

% of reinforcement required at top = $\dfrac{1.574 \times 300 \times 412.5}{100} = 1947.83mm^2$

Provide 2 × 32 Φ + 1 × 16 Φ

A_{st} provided = 1947.83 mm^2.

P_{top} provided = 1.625

Referring to table 50 of SP 16:1980

$\dfrac{M_u}{bd^2} = 4.93$

M_u cal top = 251.66 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{1.1096 \times 300 \times 412.5}{100} = 1373.13mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{bottom} provided = 1.118

Referring to table 50 of SP 16:1980

$\dfrac{M_u}{bd^2} = 3.287$

M_u cal bottom = 167.79 KN.m (Sagging)

Beam No. 243, 253:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 211.21 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$\dfrac{M_u}{bd^2} = \dfrac{211.21 \times 10^6}{300 \times 412.5^2} = 4.138$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.38 \\ P_{bottom} = 0.446 \end{array}\right] \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 133.82 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$\dfrac{M_u}{bd^2} = \dfrac{133.82 \times 10^6}{300 \times 412.5^2} = 2.622$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.893 —— —— —— —— —— $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.38

P_{bottom} = 0.893

% of reinforcement required at top = $\dfrac{1.38\times300\times412.5}{100}=1707.75mm^2$

Provide 2 × 30 Φ + 1 × 20 Φ

A_{st} provided = 1727.85 mm^2.

P_{top} provided = 1.396

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2}=4.19$$

M_u cal top = 213.88 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.893\times300\times412.5}{100}=1105.08mm^2$

Provide 2 × 25 Φ + 1 × 16 Φ

A_{st} provided = 1182.8 mm^2.

P_{bottom} provided = 0.955

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2}=2.76$$

M_u cal bottom = 140.88 KN.m (Sagging)

Beam No. 244, 254:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 160.32 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2}=\frac{160.32\times10^6}{300\times412.5^2}=3.141$$

Referring to table 50 of SP 16:1980

P_{top} = 1.073

P_{bottom} = 0.123 — — — — — $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 83.93 KN.m < $M_{u\,lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{83.93 \times 10^6}{300 \times 412.5^2} = 1.644$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.51 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.073

P_{bottom} = 0.51

% of reinforcement required at top = $\dfrac{1.073 \times 300 \times 412.5}{100} = 1327.84mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{top} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal top = 167.79 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.511 \times 300 \times 412.5}{100} = 631.11mm^2$

Provide 2 × 20 Φ + 1 × 10 Φ

A_{st} provided = 706.83 mm^2.

P_{bottom} provided = 0.571

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.816$$

M_u cal bottom = 92.7 KN.m (Sagging)

Beam No. 245, 255:

For hogging moment:

Maximum hogging moment, $M_u = 68.28$ KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{68.28 \times 10^6}{300 \times 412.5^2} = 1.3376$$

Referring to table 2 of SP 16:1980

P_{top} = 0.405 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 35.34 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{35.34 \times 10^6}{300 \times 412.5^2} = 0.692$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.2004 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.405

P_{bottom} = 0.2004

% of reinforcement required at top = $\dfrac{0.405 \times 300 \times 412.5}{100} = 501.18 mm^2$

Provide 2 × 16 Φ + 1 × 12 Φ

A_{st} provided = 515.21 mm^2.

P_{top} provided = 0.416

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.371$$

M_u cal top = 69.99 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.2004 \times 300 \times 412.5}{100} = 247.99 mm^2$

Provide 2 × 12 Φ + 1 × 10 Φ

A_{st} provided = 304.71 mm^2.

P_{bottom} provided = 0.246

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.843$$

M_u cal bottom = 43.03 KN.m (Sagging)

Beam No.246:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 269.47 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{269.47 \times 10^6}{300 \times 412.5^2} = 5.279$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.7307 \\ P_{bottom} = 0.825 \end{array}\right\} \text{— — — — —} \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 192.26 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{192.26 \times 10^6}{300 \times 412.5^2} = 3.766$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.265 \\ P_{top} = 0.325 \end{array}\right\} \text{— — — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.7307

P_{bottom} = 1.265

% of reinforcement required at top = $\dfrac{1.7307 \times 300 \times 412.5}{100} = 2141.74 mm^2$

Provide 2 × 32 Φ + 2 × 20 Φ

A_{st} provided = 2236.76 mm^2.

P_{top} provided = 1.807

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 5.526$$

M_u cal top = 282.09 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.265 \times 300 \times 412.5}{100} = 1565.43 mm^2$

Provide 2 × 25 Φ + 2 × 20 Φ

A_{st} provided = 1610.04 mm^2.

P_{bottom} provided = 1.301

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.883$$

M_u cal bottom = 198.22 KN.m (Sagging)

Beam No. 247:

For hogging moment:

Maximum hogging moment, M_u = 241.82 KN.m. > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{241.82 \times 10^6}{300 \times 412.5^2} = 4.737$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.564 \\ P_{bottom} = 0.64 \end{array}\right\} \text{— — — —} \boxed{1}$$

For sagging moment:

Maximum sagging moment = 164.44 KN.m > $M_{u\ lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{164.44 \times 10^6}{300 \times 412.5^2} = 3.221$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{bottom} = 1.098 \\ P_{top} = 0.15 \end{array}\right\} \text{— — — —} \boxed{2}$$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.564

P_{bottom} = 1.098

% of reinforcement required at top = $\frac{1.564 \times 300 \times 412.5}{100} = 1935.45 mm^2$

Provide 2 × 32 Φ + 1 × 16 Φ

A_{st} provided = 2010.66 mm^2.

P_{top} provided = 1.625

Referring to table 50 of SP 16:1980

$\frac{M_u}{bd^2} = 4.93$

M_u cal top = 251.66 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{1.098 \times 300 \times 412.5}{100} = 1358.77 mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{bottom} provided = 1.118

Referring to table 50 of SP 16:1980

$\frac{M_u}{bd^2} = 3.287$

M_u cal bottom = 167.79 KN.m (Sagging)

Beam No.248:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 209.88 KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$\frac{M_u}{bd^2} = \frac{209.88 \times 10^6}{300 \times 412.5^2} = 4.112$

Referring to table 50 of SP 16:1980

P_{top} = 1.372

P_{bottom} = 0.437 $\boxed{1}$

<u>For sagging moment:</u>

Maximum sagging moment = 132.47 KN.m < $M_{u\,lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{132.47 \times 10^6}{300 \times 412.5^2} = 2.595$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.881 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 1.372

P_{bottom} = 0.881

% of reinforcement required at top = $\frac{1.372 \times 300 \times 412.5}{100} = 1697.85 mm^2$

Provide 2 × 30 Φ + 1 × 20 Φ

A_{st} provided = 1727.85 mm^2.

P_{top} provided = 4.19

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 4.19$$

M_u cal top = 213.88 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.881 \times 300 \times 412.5}{100} = 1090.23 mm^2$

Provide 2 × 25 Φ + 1 × 12 Φ

A_{st} provided = 1094.83 mm^2.

P_{bottom} provided = 0.8847

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 2.604$$

M_u cal bottom = 132.92 KN.m (Sagging)

Beam No.249:

<u>For hogging moment:</u>

Maximum hogging moment, M_u = 157.77 KN.m. > $M_{u\,lim}$

So section is doubly reinforced.

$$\frac{M_u}{bd^2} = \frac{157.77 \times 10^6}{300 \times 412.5^2} = 3.091$$

Referring to table 50 of SP 16:1980

$$\left.\begin{array}{l} P_{top} = 1.057 \\ P_{bottom} = 0.106 \end{array}\right\} \text{— — — — —} \ \boxed{1}$$

<u>For sagging moment:</u>

Maximum sagging moment = 80.49 KN.m < $M_{u\,lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{80.49 \times 10^6}{300 \times 412.5^2} = 1.577$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.486 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 2.057

P_{bottom} = 0.486

% of reinforcement required at top = $\dfrac{1.057 \times 300 \times 412.5}{100} = 1308.03 mm^2$

Provide 2 × 25 Φ + 2 × 16 Φ

A_{st} provided = 1383.86 mm^2.

P_{top} provided = 1.118

Referring to table 50 of SP 16:1980

$$\frac{M_u}{bd^2} = 3.287$$

M_u cal top = 167.79 KN.m (Hogging)

% of reinforcement required at bottom = $\dfrac{0.486 \times 300 \times 412.5}{100} = 601.42 mm^2$

Provide 3 × 16 Φ

A_{st} provided = 603.18 mm^2.

P_{bottom} provided = 0.4874

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.581$$

M_u cal bottom = 80.71 KN.m (Sagging)

Beam No. 250:

For hogging moment:

Maximum hogging moment, $M_u = 65.18$ KN.m. < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{65.18 \times 10^6}{300 \times 412.5^2} = 1.277$$

Referring to table 2 of SP 16:1980

P_{top} = 0.385 — — — — — $\boxed{1}$

For sagging moment:

Maximum sagging moment = 29.93 KN.m < $M_{u\ lim}$

So section is singly reinforced.

$$\frac{M_u}{bd^2} = \frac{29.93 \times 10^6}{300 \times 412.5^2} = 0.586$$

Referring to table 2 of SP 16:1980

P_{bottom} = 0.168 — — — — — $\boxed{2}$

Required reinforcement is maximum of (1) & (2).

P_{top} = 0.385

P_{bottom} = 0.168

% of reinforcement required at top = $\frac{0.385 \times 300 \times 412.5}{100} = 476.43 mm^2$

Provide 2 × 16 Φ + 1 × 10 Φ

A_{st} provided = 480.65 mm^2.

P_{top} provided = 0.3884

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 1.289$$

M_u cal top = 65.8 KN.m (Hogging)

% of reinforcement required at bottom = $\frac{0.168 \times 300 \times 412.5}{100} = 207.9 mm^2$

Provide 2 × 12 Φ

A_{st} provided = 226.18 mm^2.

P_{bottom} provided = 0.1828

Referring to table 2 of SP 16:1980

$$\frac{M_u}{bd^2} = 0.636$$

M_u cal bottom = 32.47 KN.m   (Sagging)

<u>Design of columns of intermediate frame in YZ plane:</u>

Column No. 31, 56:

Axial force, P_u = 1370.9KN

Axial stress = $\frac{1370.9 \times 10^3}{400 \times 550} = 6.23 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm   ok.

$$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4 \qquad \text{ok.}$$

Assuming to use 20Φ bar with a clear cover 40mm

d^{PI} = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{550}{30} = 26.43mm$$

or 20mm which ever is greater.

So e_{min} = 26.43mm.

M_{ux} = 1370.9 × 0.02643 = 36.23KN.m

M_{ux} from analysis = 52.85KN.m

So M_{ux} = 52.85KN.m

M_{uy} = 288.39KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1370.9 \times 10^3}{20 \times 400 \times 550} = 0.311$$

Take percentage of steel, p = 3.0%

$$\frac{p}{f_{ck}} = \frac{3.0}{20} = 0.15$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.174$$

$$\therefore M_{uy1} = 0.174 \times 20 \times 550 \times 400^2 = 306.24 KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.199$$

$$\therefore M_{ux1} = 0.199 \times 20 \times 400 \times 550^2 = 481.58 KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 3.0%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.9$$

P_{uz} = 19.9 × 400 × 550 = 4378KN

$$\frac{P_u}{P_{uz}} = \frac{1370.9}{4378} = 0.313 > 0.2$$

$\therefore \alpha_n = 1.188$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{288.39}{306.24}\right)^{1.188} + \left(\frac{52.85}{481.58}\right)^{1.188} = 0.997$$

Column No. 36, 51:

Axial force, P_u = 1645.35KN

Axial stress = $\frac{1645.35 \times 10^3}{400 \times 550} = 7.47 > 0.1 f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{|}$ = 40 + 10 = 50mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{4.05 \times 10^3}{500} + \frac{550}{30} = 26.43mm$$

or 20mm which ever is greater.

So e_{min} = 26.43mm.

M_{ux} = 1645.37 × 0.02643 = 43.48KN.m

M_{ux} from analysis = 54.89KN.m

So M_{ux} = 54.89KN.m

M_{uy} = 405.29KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1645.35 \times 10^3}{20 \times 400 \times 550} = 0.374$$

Take percentage of steel, p = 3.375%

$$\frac{p}{f_{ck}} = \frac{3.375}{20} = 0.16875$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.246$$

$$\therefore M_{uy1} = 0.246 \times 20 \times 550 \times 400^2 = 422.4KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.276$$

$$\therefore M_{ux1} = 0.276 \times 20 \times 400 \times 550^2 = 667.92 KN.m$$

<u>For $P_{uz:}$</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 3.375%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 20.48$$

$P_{uz} = 20.48 \times 400 \times 550 = 4505.6$KN

$$\frac{P_u}{P_{uz}} = \frac{1645.35}{4505.6} = 0.365 > 0.2$$

$\therefore \alpha_n = 1.275$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{405.29}{422.4}\right)^{1.275} + \left(\frac{54.89}{667.92}\right)^{1.275} = 0.999$$

Column No. 41, 46:

Axial force, $P_u = 1655.6$KN

Axial stress = $\frac{1655.6 \times 10^3}{400 \times 550} = 7.52 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50$mm.

Unsupported length = 4.5 – 0.45 = 4.05m.

$$e_{min} = \frac{4.05 \times 10^3}{500} + \frac{550}{30} = 26.43mm$$

or 20mm which ever is greater.

So $e_{min} = 26.43$mm.

$M_{ux} = 1655.6 \times 0.02643 = 43.75$KN.m

M_{ux} from analysis = 55.21KN.m

So M_{ux} = 55.21KN.m

M_{uy} = 369.63KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1655.6 \times 10^3}{20 \times 400 \times 550} = 0.376$$

Take percentage of steel, p = 3.2%

$$\frac{p}{f_{ck}} = \frac{3.2}{20} = 0.16$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.22$$

$$\therefore M_{uy1} = 0.22 \times 20 \times 550 \times 400^2 = 387.2KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.276$$

$$\therefore M_{ux1} = 0.276 \times 20 \times 400 \times 550^2 = 617.1KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 3.2%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 20.2$$

P_{uz} = 20.2 × 400 × 550 = 4444KN

$$\frac{P_u}{P_{uz}} = \frac{1655.6}{4444} = 0.372 > 0.2$$

$\therefore \alpha_n = 1.287$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{369.63}{387.2}\right)^{1.287} + \left(\frac{55.21}{617.1}\right)^{1.287} = 0.996$$

Column No. 32, 57:

Axial force, P_u = 1019.62KN

Axial stress = $\frac{1019.62 \times 10^3}{400 \times 550}$ = 4.63 > $0.1f_{ck}$ = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 1019.6 × 0.02443 = 24.9KN.m

M_{ux} from analysis = 50.39KN.m

So M_{ux} = 50.39KN.m

M_{uy} = 200.58KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1019.62 \times 10^3}{20 \times 400 \times 550} = 0.232$$

Take percentage of steel, p = 2.5%

$$\frac{p}{f_{ck}} = \frac{2.5}{20} = 0.125$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.182$$

$$\therefore M_{uy1} = 0.182 \times 20 \times 550 \times 400^2 = 320.32KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.207$$

$$\therefore M_{ux1} = 0.207 \times 20 \times 400 \times 550^2 = 500.94KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.5%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 18.4$$

P_{uz} = 18.4 × 400 × 550 = 4048KN

$$\frac{P_u}{P_{uz}} = \frac{1019.62}{4048} = 0.252 > 0.2$$

$\therefore \alpha_n = 1.087$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{200.58}{320.32}\right)^{1.087} + \left(\frac{50.39}{500.94}\right)^{1.087} = 0.707$$

Column No. 37, 52:

Axial force, P_u = 1274.06KN

Axial stress = $\frac{1274.06 \times 10^3}{400 \times 550} = 5.79 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{P1} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05\times10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 1274.06 × 0.02443 = 31.12KN.m

M_{ux} from analysis = 54.13KN.m

So M_{ux} = 54.13KN.m

M_{uy} = 337.76KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{1274.06\times10^3}{20\times400\times550} = 0.29$$

Take percentage of steel, p = 2.75%

$$\frac{p}{f_{ck}} = \frac{2.75}{20} = 0.1375$$

<u>For M_{uy1}:</u>

b = 550mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.205$$

$$\therefore M_{uy1} = 0.205\times20\times550\times400^2 = 362.56KN.m$$

<u>For M_{ux1}:</u>

b = 400mm,   D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.236$$

$$\therefore M_{ux1} = 0.236\times20\times400\times550^2 = 571.12KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.75%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.6$$

$P_{uz} = 19.6 \times 400 \times 550 = 4312KN$

$$\frac{P_u}{P_{uz}} = \frac{1274.06}{4312} = 0.295 > 0.2$$

$\therefore \alpha_n = 1.158$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{337.76}{362.56}\right)^{1.158} + \left(\frac{54.13}{571.12}\right)^{1.158} = 0.991$$

Column No. 42, 47:

Axial force, $P_u = 1288.02KN$

Axial stress = $\frac{1288.02 \times 10^3}{400 \times 550} = 5.85 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50mm.$

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So $e_{min} = 24.43mm$.

$M_{ux} = 1288.02 \times 0.02443 = 31.47KN.m$

M_{ux} from analysis = 54.43KN.m

So $M_{ux} = 54.43KN.m$

$M_{uy} = 308.99KN.m$

$$\frac{P_u}{f_{ck}bd} = \frac{1288.02 \times 10^3}{20 \times 400 \times 550} = 0.293$$

Take percentage of steel, p = 2.65%

$$\frac{p}{f_{ck}} = \frac{2.65}{20} = 0.1325$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.19$$

$$\therefore M_{uy1} = 0.19 \times 20 \times 550 \times 400^2 = 334.4KN.m$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.216$$

$$\therefore M_{ux1} = 0.216 \times 20 \times 400 \times 550^2 = 522.72KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.65%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 19.3$$

$P_{uz} = 19.3 \times 400 \times 550 = 4246$KN

$$\frac{P_u}{P_{uz}} = \frac{1288.02}{4246} = 0.303 > 0.2$$

$\therefore \alpha_n = 1.171$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{308.99}{334.4}\right)^{1.171} + \left(\frac{54.31}{522.72}\right)^{1.171} = 0.984$$

Column No. 33, 58:

Axial force, P_u = 698.07KN

Axial stress = $\frac{698.07 \times 10^3}{400 \times 550} = 3.173 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4 \qquad \text{ok.}$$

Assuming to use 20Φ bar with a clear cover 40mm

$d^{P|} = 40 + 10 = 50$mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 698.07 × 0.02443 = 17.05KN.m

M_{ux} from analysis = 41.97KN.m

So M_{ux} = 41.97KN.m

M_{uy} = 193.56KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{698.07 \times 10^3}{20 \times 400 \times 550} = 0.159$$

Take percentage of steel, p = 2.2%

$$\frac{p}{f_{ck}} = \frac{2.2}{20} = 0.11$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.161$$

$$\therefore M_{uy1} = 0.161 \times 20 \times 550 \times 400^2 = 283.36KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.183$$

$\therefore M_{ux1} = 0.183 \times 20 \times 400 \times 550^2 = 442.86 KN.m$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.2%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 17.1$$

$P_{uz} = 17.1 \times 400 \times 550 = 3762$KN

$$\frac{P_u}{P_{uz}} = \frac{698.07}{3762} = 0.186 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{193.56}{283.36}\right)^{1.0} + \left(\frac{41.97}{442.86}\right)^{1.0} = 0.778$$

Column No. 38, 53:

Axial force, P_u = 918.01KN

Axial stress = $\frac{918.01 \times 10^3}{400 \times 550} = 4.17 > 0.1f_{ck} = 2$ N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{P1} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

$M_{ux} = 918.01 \times 0.02443 = 22.42$KN.m

M_{ux} from analysis = 44.62KN.m

So $M_{ux} = 44.62$KN.m

$M_{uy} = 304.31$KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{918.01\times10^3}{20\times400\times550} = 0.209$$

Take percentage of steel, p = 2.4%

$$\frac{p}{f_{ck}} = \frac{2.4}{20} = 0.12$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.188$$

$$\therefore M_{uy1} = 0.188\times20\times550\times400^2 = 330.88KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.211$$

$$\therefore M_{ux1} = 0.211\times20\times400\times550^2 = 510.62KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 2.4%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 18.4$$

$P_{uz} = 18.4 \times 400 \times 550 = 4048$KN

$$\frac{P_u}{P_{uz}} = \frac{918.01}{4048} = 0.227 > 0.2$$

$\therefore \alpha_n = 1.045$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{304.31}{330.88}\right)^{1.045} + \left(\frac{44.62}{510.62}\right)^{1.045} = 0.996$$

Column No. 43, 48:

Axial force, P_u = 930KN

Axial stress = $\frac{930 \times 10^3}{400 \times 550} = 4.227 > 0.1f_{ck}$ = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 930 × 0.02443 = 22.72KN.m

M_{ux} from analysis = 44.75KN.m

So M_{ux} = 44.75KN.m

M_{uy} = 276.71KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{930 \times 10^3}{20 \times 400 \times 550} = 0.211$$

Take percentage of steel, p = 2.275%

$$\frac{p}{f_{ck}} = \frac{2.275}{20} = 0.11375$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.171$$

$$\therefore M_{uy1} = 0.171 \times 20 \times 550 \times 400^2 = 300.96 KN.m$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.193$$

$$\therefore M_{ux1} = 0.193 \times 20 \times 400 \times 550^2 = 467.06 KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.275%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 17.6$$

P_{uz} = 17.6 × 400 × 550 = 3872KN

$$\frac{P_u}{P_{uz}} = \frac{930}{3872} = 0.24 > 0.2$$

$\therefore \alpha_n = 1.067$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{276.71}{300.96}\right)^{1.067} + \left(\frac{44.75}{467.06}\right)^{1.067} = 0.996$$

Column No. 34, 59:

Axial force, P_u = 396.84KN

Axial stress = $\frac{396.84 \times 10^3}{400 \times 550} = 1.80 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{P1} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 396.84 × 0.02443 = 9.69KN.m

M_{ux} from analysis = 33.72KN.m

So M_{ux} = 33.72KN.m

M_{uy} = 164.53KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{396.84 \times 10^3}{20 \times 400 \times 550} = 0.095$$

Take percentage of steel, p = 1.8%

$$\frac{p}{f_{ck}} = \frac{1.8}{20} = 0.09$$

<u>For M_{uy1}:</u>

b = 550mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.128$$

$$\therefore M_{uy1} = 0.128 \times 20 \times 550 \times 400^2 = 225.28KN.m$$

<u>For M_{ux1}:</u>

b = 400mm,   D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.144$$

$$\therefore M_{ux1} = 0.144 \times 20 \times 400 \times 550^2 = 348.48KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.8%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 14.8$$

P_{uz} = 14.8 × 400 × 550 = 3256KN

$$\frac{P_u}{P_{uz}} = \frac{396.84}{3256} = 0.121 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{164.53}{225.28}\right)^{1.0} + \left(\frac{33.72}{348.48}\right)^{1.0} = 0.827$$

Column No. 39, 54:

Axial force, P_u = 564.89KN

Axial stress = $\frac{564.89 \times 10^3}{400 \times 550}$ = 2.57 > 0.1f_{ck} = 2 N/ mm^2

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 564.89 × 0.02443 = 13.8KN.m

M_{ux} from analysis = 37.6KN.m

So M_{ux} = 37.6KN.m

M_{uy} = 245.54KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{564.89 \times 10^3}{20 \times 400 \times 550} = 0.128$$

Take percentage of steel, p = 2.0%

$$\frac{p}{f_{ck}} = \frac{2.0}{20} = 0.1$$

For M_{uy1}:

b = 550mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.154$$

$$\therefore M_{uy1} = 0.154 \times 20 \times 550 \times 400^2 = 271.04KN.m$$

For M_{ux1}:

b = 400mm,   D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.173$$

$$\therefore M_{ux1} = 0.173 \times 20 \times 400 \times 550^2 = 418.66KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 2.0%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 16.4$$

$P_{uz} = 16.4 \times 400 \times 550 = 3608KN$

$$\frac{P_u}{P_{uz}} = \frac{564.89}{3608} = 0.156 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{245.54}{271.04}\right)^{1.0} + \left(\frac{37.6}{418.66}\right)^{1.0} = 0.996$$

Column No. 44, 49:

Axial force, $P_u = 573.53KN$

Axial stress = $\frac{573.53 \times 10^3}{400 \times 550} = 2.61 > 0.1f_{ck} = 2\ N/mm^2$

So IS 13920:1993 specification are required.

Minimum dimension 400mm > 200mm   ok.

$\frac{b}{d} = \frac{400}{550} = 0.73 > 0.4$ ok.

Assuming to use 20Φ bar with a clear cover 40mm

$d^{Pl} = 40 + 10 = 50mm.$

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 573.53 × 0.02443 = 14.01KN.m

M_{ux} from analysis = 37.67KN.m

So M_{ux} = 37.67KN.m

M_{uy} = 228.77KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{573.53 \times 10^3}{20 \times 400 \times 550} = 0.13$$

Take percentage of steel, p = 1.9%

$$\frac{p}{f_{ck}} = \frac{1.9}{20} = 0.095$$

For M_{uy1}:

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.144$$

$$\therefore M_{uy1} = 0.144 \times 20 \times 550 \times 400^2 = 253.44KN.m$$

For M_{ux1}:

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.162$$

$$\therefore M_{ux1} = 0.162 \times 20 \times 400 \times 550^2 = 392.04KN.m$$

For $P_{uz:}$

Referring to chart 63 SP 16:1980

Corresponding to p = 1.9%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 15.9$$

P_{uz} = 15.9 × 400 × 550 = 3498KN

$$\frac{P_u}{P_{uz}} = \frac{573.53}{3498} = 0.164 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{228.77}{253.44}\right)^{1.0} + \left(\frac{37.67}{392.04}\right)^{1.0} = 0.998$$

Column No. 35, 60:

Axial force, P_u = 143.92KN

Axial stress = $\frac{143.92 \times 10^3}{400 \times 550} = 0.654 < 0.1 f_{ck} = 2 \text{ N/ mm}^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{P1} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 143.92 × 0.02443 = 3.52KN.m

M_{ux} from analysis = 25.84KN.m

So M_{ux} = 25.84KN.m

M_{uy} = 105.24KN.m

$$\frac{P_u}{f_{ck} bd} = \frac{143.92 \times 10^3}{20 \times 400 \times 550} = 0.033$$

Take percentage of steel, p = 1.375%

$$\frac{p}{f_{ck}} = \frac{1.375}{20} = 0.06875$$

For M_{uy1}:

b = 550mm,   D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.099$$

$$\therefore M_{uy1} = 0.099 \times 20 \times 550 \times 400^2 = 174.24 KN.m$$

For M_{ux1}:

b = 400mm,   D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.11$$

$$\therefore M_{ux1} = 0.11 \times 20 \times 400 \times 550^2 = 266.2 KN.m$$

For P_{uz}:

Referring to chart 63 SP 16:1980

Corresponding to p = 1.375%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.3$$

P_{uz} = 13.3 × 400 × 550 = 2926KN

$$\frac{P_u}{P_{uz}} = \frac{143.92}{2926} = 0.049 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{105.24}{174.24}\right)^{1.0} + \left(\frac{25.84}{266.2}\right)^{1.0} = 0.701$$

Column No. 40, 55:

Axial force, P_u = 214.26KN

Axial stress = $\frac{214.26 \times 10^3}{400 \times 550} = 0.974 < 0.1 f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05\times10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 214.26 × 0.02443 = 5.23KN.m

M_{ux} from analysis = 23.56KN.m

So M_{ux} = 23.56KN.m

M_{uy} = 165.72KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{214.26\times10^3}{20\times400\times550} = 0.049$$

Take percentage of steel, p = 1.6%

$$\frac{p}{f_{ck}} = \frac{1.6}{20} = 0.08$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.103$$

$$\therefore M_{uy1} = 0.103\times20\times550\times400^2 = 181.28KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.115$$

$$\therefore M_{ux1} = 0.115\times20\times400\times550^2 = 278.3KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.6%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.5$$

P_{uz} = 13.5 × 400 × 550 = 2970KN

$$\frac{P_u}{P_{uz}} = \frac{214.26}{2970} = 0.072 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{165.72}{181.28}\right)^{1.0} + \left(\frac{23.56}{278.3}\right)^{1.0} = 0.998$$

Column No. 45, 50:

Axial force, P_u = 218.1KN

Axial stress = $\frac{218.1 \times 10^3}{400 \times 550} = 0.991 < 0.1f_{ck} = 2\ N/mm^2$

Assuming to use 20Φ bar with a clear cover 40mm

d^{Pl} = 40 + 10 = 50mm.

Unsupported length = 3.5 – 0.45 = 4.05m.

$$e_{\min} = \frac{3.05 \times 10^3}{500} + \frac{550}{30} = 24.43mm$$

or 20mm which ever is greater.

So e_{min} = 24.43mm.

M_{ux} = 218.1 × 0.02443 = 5.33KN.m

M_{ux} from analysis = 23.52KN.m

So M_{ux} = 23.52KN.m

M_{uy} = 152.27KN.m

$$\frac{P_u}{f_{ck}bd} = \frac{218.1 \times 10^3}{20 \times 400 \times 550} = 0.0496$$

Take percentage of steel, p = 1.425%

$$\frac{p}{f_{ck}} = \frac{1.425}{20} = 0.07125$$

<u>For M_{uy1}:</u>

b = 550mm, D = 400mm, $\frac{d'}{D} = \frac{50}{400} = 0.125$

Referring chart 45 of SP 16:1980

$$\frac{M_{uy1}}{f_{ck}bd^2} = 0.096$$

$$\therefore M_{uy1} = 0.096 \times 20 \times 550 \times 400^2 = 168.96KN.m$$

<u>For M_{ux1}:</u>

b = 400mm, D = 550mm, $\frac{d'}{D} = \frac{50}{550} = 0.09$

Referring chart 44 of SP 16:1980

$$\frac{M_{ux1}}{f_{ck}bd^2} = 0.106$$

$$\therefore M_{ux1} = 0.106 \times 20 \times 400 \times 550^2 = 256.52KN.m$$

<u>For P_{uz}:</u>

Referring to chart 63 SP 16:1980

Corresponding to p = 1.425%, M20 & Fe415

$$\frac{P_{uz}}{A_g} = 13.1$$

P_{uz} = 13.1 × 400 × 550 = 2882KN

$$\frac{P_u}{P_{uz}} = \frac{218.1}{2882} = 0.076 < 0.2$$

$\therefore \alpha_n = 1.0$

$$\therefore \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} + \left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} = \left(\frac{152.27}{168.96}\right)^{1.0} + \left(\frac{23.52}{256.52}\right)^{1.0} = 0.992$$

<u>**Design for shear reinforcement for beams of end frame in XZ plane:**</u>

Beam No. 125, 135:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (5 + 3.375) × 4 + (0.3 × 3.5 × 4) = 37.7KN

$$V_{AS1} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{55.85 + 96.58}{4}\right) = -30.73KN$$

$$V_{BS1} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{55.85 + 96.58}{4}\right) = 75.97KN$$

$$V_{AS2} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{55.85 + 96.58}{4}\right) = 75.97KN$$

$$V_{BS2} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{55.85 + 96.58}{4}\right) = -30.73KN$$

Shear force from analysis =74.18KN

$\therefore V_u = 75.97KN$

$$\tau_v = \frac{V_u}{bd} = \frac{75.97 \times 10^3}{300 \times 412.5} = 0.614 N/mm^2$$

$P_t = 0.289$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.379 \text{ N/mm}^2$

$\tau_{c\ max} = 2.8 \text{ N/mm}^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 75.97 \times 10^3 - 0.379 \times 300 \times 412.5$

$= 29.06 \times 10^3 \text{N/mm}^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{29.06 \times 10^3}$$

$= 512.5mm$

According to IS 13920:1993

$S_{v\ min} = 100\text{mm}$

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128\text{mm}$

iii) 512.5mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No.130:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (3.75 + 3.375) × 3 + (0.3 × 2.625 × 3) = 23.73KN

$$V_{AS1} = 1.2 \times \left(\frac{23.73}{2}\right) - 1.4 \times \left(\frac{66.99 + 55.85}{3}\right) = -44.48KN$$

$$V_{BS1} = 1.2 \times \left(\frac{23.73}{2}\right) + 1.4 \times \left(\frac{66.99 + 55.85}{3}\right) = 72.96KN$$

$$V_{AS2} = 1.2 \times \left(\frac{23.73}{2}\right) + 1.4 \times \left(\frac{66.99 + 55.85}{3}\right) = 72.96KN$$

$$V_{BS2} = 1.2 \times \left(\frac{23.73}{2}\right) - 1.4 \times \left(\frac{66.99 + 55.85}{3}\right) = -44.48KN$$

Shear force from analysis =51.57KN

$\therefore V_u$ = 72.96KN

$$\tau_v = \frac{V_u}{bd} = \frac{72.96 \times 10^3}{300 \times 412.5} = 0.59N / mm^2$$

P_t = 0.325

Refering table 19 of IS 456: 2000

$\therefore \tau_c$ = 0.396 N/mm^2

$\tau_{c\,max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 72.96 \times 10^3 - 0.396 \times 300 \times 412.5$

$= 23.96 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{23.96 \times 10^3}$$

$$= 621.59mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 621.59mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 124, 134:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{118.05 + 198.22}{4}\right) = -55.07KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{118.05 + 198.22}{4}\right) = 166.31KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{118.05 + 198.22}{4}\right) = 166.31KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{118.05 + 198.22}{4}\right) = -55.07KN$$

Shear force from analysis =137.52KN

$\therefore V_u = 166.31$KN

$$\tau_v = \frac{V_u}{bd} = \frac{166.31 \times 10^3}{300 \times 412.5} = 1.344 N/mm^2$$

$P_t = 0.762$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.563$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 166.31 \times 10^3 - 0.563 \times 300 \times 412.5$

$= 96.64 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{96.64 \times 10^3}$$

$$= 154.11mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 154.11mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 129:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (17.875 + 3.375) × 3 + (0.3 × 2.625 × 3) = 66.11KN

$$V_{AS1} = 1.2 \times \left(\frac{66.11}{2}\right) - 1.4 \times \left(\frac{140.89 + 198.22}{3}\right) = -118.58KN$$

$$V_{BS1} = 1.2 \times \left(\frac{66.11}{2}\right) + 1.4 \times \left(\frac{140.89 + 198.22}{3}\right) = 197.91KN$$

$$V_{AS2} = 1.2 \times \left(\frac{66.11}{2}\right) + 1.4 \times \left(\frac{140.89 + 198.22}{3}\right) = 197.91KN$$

$$V_{BS2} = 1.2 \times \left(\frac{66.11}{2}\right) - 1.4 \times \left(\frac{140.89 + 198.22}{3}\right) = -118.58KN$$

Shear force from analysis =148.75KN

∴ V_u = 197.9KN

$$\tau_v = \frac{V_u}{bd} = \frac{197.9 \times 10^3}{300 \times 412.5} = 1.599 N/mm^2$$

P_t = 0.955

Refering table 19 of IS 456: 2000

∴ τ_c = 0.609 N/mm²

$\tau_{c\,max}$ = 2.8 N/mm² for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 197.91 \times 10^3 - 0.609 \times 300 \times 412.5$

$= 122.54 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{122.54 \times 10^3}$$

$$= 121.54 mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 121.54mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 123, 133:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{179.84 + 256.06}{4}\right) = -96.94KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{179.84 + 256.06}{4}\right) = 208.18KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{179.84 + 256.06}{4}\right) = 208.18KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{179.84 + 256.06}{4}\right) = -96.94KN$$

Shear force from analysis =168.42KN

$\therefore V_u$ = 208.18KN

$$\tau_v = \frac{V_u}{bd} = \frac{208.18 \times 10^3}{300 \times 412.5} = 1.682 N/mm^2$$

P_t = 1.19

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.658 \text{ N/mm}^2$

$\tau_{c\,max} = 2.8 \text{ N/mm}^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 218.18 \times 10^3 - 0.658 \times 300 \times 412.5$

$= 126.75 \times 10^3 \text{N/mm}^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{126.75 \times 10^3}$$

$$= 117.5mm$$

According to IS 13920:1993

$S_{v\,min} = 100\text{mm}$

$S_{v\,max}$ = i) $\dfrac{d}{4} = \dfrac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 25 = 200\text{mm}$

iii) 117.5mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 128:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (17.875 + 3.375) × 3 + (0.3 × 2.625 × 3) = 66.11KN

$$V_{AS1} = 1.2 \times \left(\frac{66.11}{2}\right) - 1.4 \times \left(\frac{225.81 + 266.22}{3}\right) = -189.9KN$$

$$V_{BS1} = 1.2 \times \left(\frac{66.11}{2}\right) + 1.4 \times \left(\frac{225.81 + 266.22}{3}\right) = 269.2KN$$

$$V_{AS2} = 1.2 \times \left(\frac{66.11}{2}\right) + 1.4 \times \left(\frac{225.81 + 266.22}{3}\right) = 269.2KN$$

$$V_{BS2} = 1.2 \times \left(\frac{66.11}{2}\right) - 1.4 \times \left(\frac{225.81 + 266.22}{3}\right) = -189.9KN$$

Shear force from analysis =204.05KN

$\therefore V_u = 269.2KN$

$$\tau_v = \frac{V_u}{bd} = \frac{269.2 \times 10^3}{300 \times 412.5} = 2.17 N/mm^2$$

$P_t = 1.467$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.7134\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 269.2 \times 10^3 - 0.7134 \times 300 \times 412.5$

$= 180.92 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{180.92 \times 10^3}$$

$$= 82.32 mm$$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13 mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128mm$

iii) 82.32mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 122, 132:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = -125.46 KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = 236.69 KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = 236.69KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = -125.46KN$$

Shear force from analysis =185.30KN

$\therefore V_u = 236.69KN$

$$\tau_v = \frac{V_u}{bd} = \frac{236.69 \times 10^3}{300 \times 412.5} = 1.91N/mm^2$$

$P_t = 1.396$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.6992\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 236.69 \times 10^3 - 0.6992 \times 300 \times 412.5$

$= 150.16 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{150.16 \times 10^3}$$

$$= 99.18mm$$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160mm$

iii) 99.18mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 127:

Self wt of beam = $0.3 \times 0.45 \times 25 = 3.375KN/m$

WL = (17.875 + 3.375) × 3 + (0.3 × 2.625 × 3) = 66.11KN

$$V_{AS1} = 1.2\times\left(\frac{66.11}{2}\right) - 1.4\times\left(\frac{282.09+329.5}{3}\right) = -243.11KN$$

$$V_{BS1} = 1.2\times\left(\frac{66.11}{2}\right) + 1.4\times\left(\frac{282.09+329.5}{3}\right) = 312.44KN$$

$$V_{AS2} = 1.2\times\left(\frac{66.11}{2}\right) + 1.4\times\left(\frac{282.09+329.5}{3}\right) = 312.44KN$$

$$V_{BS2} = 1.2\times\left(\frac{66.11}{2}\right) - 1.4\times\left(\frac{282.09+329.5}{3}\right) = -243.11KN$$

Shear force from analysis =242.29KN

$\therefore V_u = 312.44KN$

$$\tau_v = \frac{V_u}{bd} = \frac{312.44\times10^3}{300\times412.5} = 2.52N/mm^2$$

$P_t = 1.864$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.768\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 312.44 \times 10^3 – 0.768 \times 300 \times 412.5$

$= 217.4 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87\times f_y\times A_{SV}\times d}{V_{us}}$$

$$= \frac{0.87\times415\times(2\times50)\times412.5}{217.4\times10^3}$$

$= 68.51mm$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 25 = 200mm$

iii) 68.51mm

Provide 8Φ two legged stirrup @ 65mm c/c.

Beam No. 121, 131:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = -147.8KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = 259.03KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = 259.03KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = -147.8KN$$

Shear force from analysis =201.2KN

$\therefore V_u = 259.03KN$

$$\tau_v = \frac{V_u}{bd} = \frac{259.03 \times 10^3}{300 \times 412.5} = 2.09 N/mm^2$$

$P_t = 1.65$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.738\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 259.03 \times 10^3 - 0.738 \times 300 \times 412.5$

$= 167.7 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{167.7 \times 10^3}$$

$$= 88.8mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\dfrac{d}{4} = \dfrac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 88.8mm

Provide 8Φ two legged stirrup @ 85mm c/c.

Beam No. 126:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (17.875 + 3.375) × 3 + (0.3 × 2.625 × 3) = 66.11KN

$$V_{AS1} = 1.2 \times \left(\frac{66.11}{2}\right) - 1.4 \times \left(\frac{303.47 + 360.9}{3}\right) = -273.37KN$$

$$V_{BS1} = 1.2 \times \left(\frac{66.11}{2}\right) + 1.4 \times \left(\frac{303.47 + 360.9}{3}\right) = 331.04KN$$

$$V_{AS2} = 1.2 \times \left(\frac{66.11}{2}\right) + 1.4 \times \left(\frac{303.47 + 360.9}{3}\right) = 331.04KN$$

$$V_{BS2} = 1.2 \times \left(\frac{66.11}{2}\right) - 1.4 \times \left(\frac{303.47 + 360.9}{3}\right) = -273.37KN$$

Shear force from analysis =262.62KN

$\therefore V_u$ = 331.04KN

$$\tau_v = \frac{V_u}{bd} = \frac{331.04 \times 10^3}{300 \times 412.5} = 2.67N/mm^2$$

P_t = 1.975

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.786$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 331.04 \times 10^3 - 0.768 \times 300 \times 412.5$

$= 233.77 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{233.77 \times 10^3}$$

$$= 63.71mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) 8 × Φ_{min} = 8 × 20 = 160mm

iii) 63.71mm

Provide 8Φ two legged stirrup @ 60mm c/c.

<u>Design for shear reinforcement for beams of intermediate frame in XZ plane:</u>

Beam No. 140, 150:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (10 + 3.375) × 4 + (0.3 × 7 × 4) = 61.9KN

$$V_{AS1} = 1.2 \times \left(\frac{61.9}{2}\right) - 1.4 \times \left(\frac{47.52 + 103.37}{4}\right) = -15.67KN$$

$$V_{BS1} = 1.2 \times \left(\frac{61.9}{2}\right) + 1.4 \times \left(\frac{47.52 + 103.37}{4}\right) = 89.95KN$$

$$V_{AS2} = 1.2 \times \left(\frac{61.9}{2}\right) + 1.4 \times \left(\frac{47.52 + 103.37}{4}\right) = 89.95KN$$

$$V_{BS2} = 1.2 \times \left(\frac{61.9}{2}\right) - 1.4 \times \left(\frac{47.52 + 103.37}{4}\right) = -15.67KN$$

Shear force from analysis =74.78KN

∴ V_u = 89.95KN

$$\tau_v = \frac{V_u}{bd} = \frac{89.95 \times 10^3}{300 \times 412.5} = 0.726N/mm^2$$

P_t = 0.274

Refering table 19 of IS 456: 2000

∴ τ_c = 0.372 N/mm^2

$\tau_{c\,max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 89.95 \times 10^3 - 0.372 \times 300 \times 412.5$

$= 43.915 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{43.915 \times 10^3}$$

$$= 339.14mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 339.14mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 139, 149:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.5 + 3.375) × 4 + (0.3 × 7 × 4) = 110.9KN

$$V_{AS1} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{108.86 + 202.14}{4}\right) = -42.31KN$$

$$V_{BS1} = 1.2 \times \left(\frac{110.9}{2}\right) + 1.4 \times \left(\frac{108.86 + 202.14}{4}\right) = 175.39KN$$

$$V_{AS2} = 1.2 \times \left(\frac{110.9}{2}\right) + 1.4 \times \left(\frac{108.86 + 202.14}{4}\right) = 175.39KN$$

$$V_{BS2} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{108.86 + 202.14}{4}\right) = -42.31KN$$

Shear force from analysis =148.99KN

$\therefore V_u$ = 175.39KN

$$\tau_v = \frac{V_u}{bd} = \frac{175.39\times10^3}{300\times412.5} = 1.417 N/mm^2$$

$P_t = 0.69$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.541$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 175.39 \times 10^3 - 0.541 \times 300 \times 412.5$

$= 108.44 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87\times f_y \times A_{SV}\times d}{V_{us}}$$

$$= \frac{0.87\times415\times(2\times50)\times412.5}{108.44\times10^3}$$

$$= 137.34mm$$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 137.34mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 138, 148:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.5 + 3.375) × 4 + (0.3 × 7 × 4) = 110.9KN

$$V_{AS1} = 1.2\times\left(\frac{110.9}{2}\right) - 1.4\times\left(\frac{110.9+266.22}{4}\right) = -89.58KN$$

$$V_{BS1} = 1.2\times\left(\frac{110.9}{2}\right) + 1.4\times\left(\frac{110.9+266.22}{4}\right) = 222.66KN$$

$$V_{AS2} = 1.2\times\left(\frac{110.9}{2}\right) + 1.4\times\left(\frac{110.9+266.22}{4}\right) = 222.66KN$$

$$V_{BS2} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{110.9 + 266.22}{4}\right) = -89.58KN$$

Shear force from analysis =179.89KN

$\therefore$ V_u = 222.66KN

$$\tau_v = \frac{V_u}{bd} = \frac{222.66 \times 10^3}{300 \times 412.5} = 1.799 N/mm^2$$

P_t = 1.19

Refering table 19 of IS 456: 2000

$\therefore$ τ_c = 0.658 N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 222.66 \times 10^3 - 0.658 \times 300 \times 412.5$

$= 141.23 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{141.23 \times 10^3}$$

$$= 105.45mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 25 = 200$mm

iii) 105.58mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 137, 147:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.5 + 3.375) × 4 + (0.3 × 7 × 4) = 110.9KN

$$V_{AS1} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = -114.84KN$$

$$V_{BS1} = 1.2 \times \left(\frac{110.9}{2}\right) + 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = 247.32KN$$

$$V_{AS2} = 1.2 \times \left(\frac{110.9}{2}\right) + 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = 247.32KN$$

$$V_{BS2} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{213.89 + 303.48}{4}\right) = -114.84KN$$

Shear force from analysis =196.67KN

$\therefore V_u = 247.32KN$

$$\tau_v = \frac{V_u}{bd} = \frac{247.32 \times 10^3}{300 \times 412.5} = 1.998N/mm^2$$

$P_t = 1.396$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.699\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 247.33 \times 10^3 – 0.699 \times 300 \times 412.5$

$= 160.82 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{160.82 \times 10^3}$$

$= 92.60mm$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160mm$

iii) 92.6mm

Provide 8Φ two legged stirrup @ 90mm c/c.

Beam No. 136, 146:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.5 + 3.375) × 4 + (0.3 × 7 × 4) = 110.9KN

$$V_{AS1} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = -136.87KN$$

$$V_{BS1} = 1.2 \times \left(\frac{110.9}{2}\right) + 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = 269.95KN$$

$$V_{AS2} = 1.2 \times \left(\frac{110.9}{2}\right) + 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = 269.95KN$$

$$V_{BS2} = 1.2 \times \left(\frac{110.9}{2}\right) - 1.4 \times \left(\frac{251.66 + 329.5}{4}\right) = -136.87KN$$

Shear force from analysis =212.26KN

$\therefore V_u = 269.95KN$

$$\tau_v = \frac{V_u}{bd} = \frac{269.95 \times 10^3}{300 \times 412.5} = 2.18N/mm^2$$

$P_t = 1.625$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.735\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 269.95 \times 10^3 - 0.735 \times 300 \times 412.5$

$= 178.99 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{178.99 \times 10^3}$$

$$= 83.2mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 83.2mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 145:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (7.5 + 3.375) × 3 + (0.3 × 5.25 × 3) = 37.35KN

$$V_{AS1} = 1.2 \times \left(\frac{37.35}{2}\right) - 1.4 \times \left(\frac{47.52 + 69.99}{3}\right) = -32.43KN$$

$$V_{BS1} = 1.2 \times \left(\frac{37.35}{2}\right) + 1.4 \times \left(\frac{47.52 + 69.99}{3}\right) = 77.35KN$$

$$V_{AS2} = 1.2 \times \left(\frac{37.35}{2}\right) + 1.4 \times \left(\frac{47.52 + 69.99}{3}\right) = 77.35KN$$

$$V_{BS2} = 1.2 \times \left(\frac{37.35}{2}\right) - 1.4 \times \left(\frac{47.52 + 69.99}{3}\right) = -32.43KN$$

Shear force from analysis =60.12KN

$\therefore V_u$ = 77.25KN

$$\tau_v = \frac{V_u}{bd} = \frac{77.25 \times 10^3}{300 \times 412.5} = 0.624N/mm^2$$

P_t = 0.274

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.372$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 77.25 \times 10^3 - 0.372 \times 300 \times 412.5$

$= 31.22 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{31.22 \times 10^3}$$

$$= 477mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 477mm

Provide 8Φ two legged stirrup @ 95mm c/c

Beam No. 144:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (20.5 + 3.375) × 3 + (0.3 × 5.25 × 3) = 76.35KN

$$V_{AS1} = 1.2 \times \left(\frac{76.35}{2}\right) - 1.4 \times \left(\frac{139.93 + 198.22}{3}\right) = -108.73KN$$

$$V_{BS1} = 1.2 \times \left(\frac{76.35}{2}\right) + 1.4 \times \left(\frac{139.93 + 198.22}{3}\right) = 200.35KN$$

$$V_{AS2} = 1.2 \times \left(\frac{76.35}{2}\right) + 1.4 \times \left(\frac{139.93 + 198.22}{3}\right) = 200.35KN$$

$$V_{BS2} = 1.2 \times \left(\frac{76.35}{2}\right) - 1.4 \times \left(\frac{139.93 + 198.22}{3}\right) = -108.73KN$$

Shear force from analysis =155.18KN

$\therefore V_u$ = 200.35KN

$$\tau_v = \frac{V_u}{bd} = \frac{200.35 \times 10^3}{300 \times 412.5} = 1.619N/mm^2$$

P_t = 0.8847

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.592$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 200.35 \times 10^3 - 0.592 \times 300 \times 412.5$

$= 127.09 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{127.09 \times 10^3}$$

$$= 117mm$$

According to IS 13920:1993

$S_{v\,min} = 100mm$

$S_{v\,max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96mm$

iii) 117mm

Provide 8Φ two legged stirrup @ 95mm c/c

Beam No. 143:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (20.5 + 3.375) × 3 + (0.3 × 5.25 × 3) = 76.35KN

$$V_{AS1} = 1.2 \times \left(\frac{76.35}{2}\right) - 1.4 \times \left(\frac{225.8 + 282.09}{3}\right) = -180.09KN$$

$$V_{BS1} = 1.2 \times \left(\frac{76.35}{2}\right) + 1.4 \times \left(\frac{225.8 + 282.09}{3}\right) = 261.66KN$$

$$V_{AS2} = 1.2 \times \left(\frac{76.35}{2}\right) + 1.4 \times \left(\frac{225.8 + 282.09}{3}\right) = 261.66KN$$

$$V_{BS2} = 1.2 \times \left(\frac{76.35}{2}\right) - 1.4 \times \left(\frac{225.8 + 282.09}{3}\right) = -180.09KN$$

Shear force from analysis =210.67KN

$\therefore V_u = 261.66KN$

$$\tau_v = \frac{V_u}{bd} = \frac{261.66 \times 10^3}{300 \times 412.5} = 2.11 N/mm^2$$

$P_t = 1.4673$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.713$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 261.66 \times 10^3 - 0.713 \times 300 \times 412.5$

$= 173.42 \times 10^3$ N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{173.42 \times 10^3}$$

$= 85.88 mm$

According to IS 13920:1993

$S_{v\,min} = 100$mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13 mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 85.88mm

Provide 8Φ two legged stirrup @ 85mm c/c

Beam No. 142:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (20.5 + 3.375) × 3 + (0.3 × 5.25 × 3) = 76.35KN

$$V_{AS1} = 1.2 \times \left(\frac{76.35}{2}\right) - 1.4 \times \left(\frac{282.09 + 329.5}{3}\right) = -225.41 KN$$

$$V_{BS1} = 1.2 \times \left(\frac{76.35}{2}\right) + 1.4 \times \left(\frac{282.09 + 329.5}{3}\right) = 317.03 KN$$

$$V_{AS2} = 1.2 \times \left(\frac{76.35}{2}\right) + 1.4 \times \left(\frac{282.09 + 329.5}{3}\right) = 317.03KN$$

$$V_{BS2} = 1.2 \times \left(\frac{76.35}{2}\right) - 1.4 \times \left(\frac{282.09 + 329.5}{3}\right) = -225.41KN$$

Shear force from analysis =249.02KN

$\therefore V_u = 317.03$KN

$$\tau_v = \frac{V_u}{bd} = \frac{317.03 \times 10^3}{300 \times 412.5} = 2.56N/mm^2$$

$P_t = 1.807$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.759$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 317.03 \times 10^3 - 0.759 \times 300 \times 412.5$

$= 223.10 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{223.10 \times 10^3}$$

$= 66.75mm$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 66.75mm

Provide 8Φ two legged stirrup @ 65mm c/c

Beam No. 141:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (20.5 + 3.375) × 3 + (0.3 × 5.25 × 3) = 76.35KN

$$V_{AS1} = 1.2 \times \left(\frac{76.35}{2} \right) - 1.4 \times \left(\frac{303.47 + 360.9}{3} \right) = -247.23KN$$

$$V_{BS1} = 1.2 \times \left(\frac{76.35}{2} \right) + 1.4 \times \left(\frac{303.47 + 360.9}{3} \right) = 336.85KN$$

$$V_{AS2} = 1.2 \times \left(\frac{76.35}{2} \right) + 1.4 \times \left(\frac{303.47 + 360.9}{3} \right) = 336.85KN$$

$$V_{BS2} = 1.2 \times \left(\frac{76.35}{2} \right) - 1.4 \times \left(\frac{303.47 + 360.9}{3} \right) = -247.23KN$$

Shear force from analysis =269.38KN

$\therefore V_u = 336.85$KN

$$\tau_v = \frac{V_u}{bd} = \frac{336.85 \times 10^3}{300 \times 412.5} = 2.72 N/mm^2$$

$P_t = 1.936$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.742$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 336.85 \times 10^3 - 0.742 \times 300 \times 412.5$

$= 245.03 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{245.03 \times 10^3}$$

$= 60.29mm$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 25 = 200$mm

iii) 60.29mm

Provide 8Φ two legged stirrup @ 60mm c/c

<u>Design for shear reinforcement for beams of end frame in YZ plane:</u>

Beam No. 211, 231:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{256.05 + 309.09}{4}\right) = -142.18KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{256.05 + 309.09}{4}\right) = 253.42KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{256.05 + 309.09}{4}\right) = 253.42KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{256.05 + 309.09}{4}\right) = -142.18KN$$

Shear force from analysis = 187.08 KN

$\therefore V_u = 253.42KN$

$$\tau_v = \frac{V_u}{bd} = \frac{253.42 \times 10^3}{300 \times 412.5} = 2.048N/mm^2$$

$P_t = 1.65$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.738 \text{ N/mm}^2$

$\tau_{c\,max} = 2.8 \text{ N/mm}^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 253.42 \times 10^3 - 0.738 \times 300 \times 412.5$

$= 162.09 \times 10^3 \text{N/mm}^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{162.09 \times 10^3}$$

$$= 91.88mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 91.88mm

Provide 8Φ two legged stirrup @ 90mm c/c.

Beam No. 212, 232:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = -107.07KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = 218.31KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = 218.31KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = -107.07KN$$

Shear force from analysis = 161.86 KN

$\therefore V_u$ = 218.31KN

$$\tau_v = \frac{V_u}{bd} = \frac{218.31 \times 10^3}{300 \times 412.5} = 1.764 N/mm^2$$

P_t = 1.301

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.68$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 218.31 \times 10^3 - 0.68 \times 300 \times 412.5$

$= 134.16 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{134.16 \times 10^3}$$

$$= 111.01mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 111.01mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 213, 233:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = -86.26KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = 197.5KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = 197.5KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = -86.26KN$$

Shear force from analysis = 145.9 KN

∴ V_u = 197.51KN

$$\tau_v = \frac{V_u}{bd} = \frac{197.5 \times 10^3}{300 \times 412.5} = 1.596N/mm^2$$

P_t = 1.118

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.644$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 197.5 \times 10^3 - 0.644 \times 300 \times 412.5$

$= 117.81 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{1117.81 \times 10^3}$$

$$= 126.42mm$$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 126.42mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 214, 234:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{103.37 + 167.79}{4}\right) = -39.29KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{103.37 + 167.79}{4}\right) = 150.53KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{103.37 + 167.79}{4}\right) = 150.53KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{103.37 + 167.79}{4}\right) = -39.29KN$$

Shear force from analysis = 115.49 KN

$\therefore V_u = 150.53KN$

$$\tau_v = \frac{V_u}{bd} = \frac{150.53\times10^3}{300\times412.5} = 1.216N/mm^2$$

$P_t = 0.649$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.528\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 150.53\times10^3 - 0.529\times300\times412.5$

$= 85.19\times10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87\times f_y\times A_{SV}\times d}{V_{us}}$$

$$= \frac{0.87\times415\times(2\times50)\times412.5}{85.19\times10^3}$$

$$= 174.82mm$$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8\times\Phi_{min} = 8\times16 = 128mm$

iii) 174.82mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 215, 235:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (5 + 3.375) × 4 + (0.3 × 3.5 × 4) = 37.7KN

$$V_{AS1} = 1.2\times\left(\frac{37.7}{2}\right) - 1.4\times\left(\frac{43.03+69.99}{4}\right) = -16.93KN$$

$$V_{BS1} = 1.2\times\left(\frac{37.7}{2}\right) + 1.4\times\left(\frac{43.03+69.99}{4}\right) = 62.18KN$$

$$V_{AS2} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{43.03 + 69.99}{4}\right) = 62.18KN$$

$$V_{BS2} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{43.03 + 69.99}{4}\right) = -16.93KN$$

Shear force from analysis = 53.25 KN

$\therefore$ V_u = 62.18KN

$$\tau_v = \frac{V_u}{bd} = \frac{62.18 \times 10^3}{300 \times 412.5} = 0.502 N/mm^2$$

P_t = 0.246

Refering table 19 of IS 456: 2000

$\therefore$ $\tau_c = 0.357$ N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 62.18 \times 10^3 - 0.357 \times 300 \times 412.5$

$= 18 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{18 \times 10^3}$$

$$= 827mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 10 = 80$mm

iii) 827mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 216, 226:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{198.22 + 256.05}{4}\right) = -103.37KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{198.22 + 256.05}{4}\right) = 214.61KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{198.22 + 256.05}{4}\right) = 214.61KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{198.22 + 256.05}{4}\right) = -103.37KN$$

Shear force from analysis = 160.68 KN

$\therefore V_u = 214.61KN$

$$\tau_v = \frac{V_u}{bd} = \frac{214.61 \times 10^3}{300 \times 412.5} = 1.734 N/mm^2$$

$P_t = 1.301$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.68\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 214.61 \times 10^3 - 0.68 \times 300 \times 412.5$

$= 130.46 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{130.46 \times 10^3}$$

$$= 114.16mm$$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160mm$

iii) 174.82mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 217, 227:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = -86.26KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = 197.5KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = 197.5KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = -86.26KN$$

Shear force from analysis = 151.28 KN

$\therefore V_u = 197.5KN$

$$\tau_v = \frac{V_u}{bd} = \frac{197.5 \times 10^3}{300 \times 412.5} = 1.596 N/mm^2$$

$P_t = 1.118$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.644 \ N/mm^2$

$\tau_{c\,max} = 2.8 \ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 197.5 \times 10^3 - 0.644 \times 300 \times 412.5$

$= 117.81 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{117.81 \times 10^3}$$

$$= 126.42mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 126.42mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 218, 228:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = -60.28KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = 171.52KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = 171.52KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = -60.28KN$$

Shear force from analysis = 136.12 KN

$\therefore V_u$ = 171.52KN

$$\tau_v = \frac{V_u}{bd} = \frac{171.52 \times 10^3}{300 \times 412.5} = 1.386 N/mm^2$$

P_t = 0.8847

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.592$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 171.52 \times 10^3 - 0.592 \times 300 \times 412.5$

$= 98.26 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{98.26 \times 10^3}$$

$$= 151.57mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 174.82mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 219, 229:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{92.7 + 150.84}{4}\right) = -29.62KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{92.7 + 150.84}{4}\right) = 140.86KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{92.7 + 150.84}{4}\right) = 140.86KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{92.7 + 150.84}{4}\right) = -29.62KN$$

Shear force from analysis = 112.11 KN

$\therefore V_u$ = 140.86KN

$$\tau_v = \frac{V_u}{bd} = \frac{140.86 \times 10^3}{300 \times 412.5} = 1.138N/mm^2$$

P_t = 0.571

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.503$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 140.86 \times 10^3 - 0.503 \times 300 \times 412.5$

$= 78.61 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{78.61 \times 10^3}$$

$$= 189.46mm$$

According to IS 13920:1993

$S_{v\,min} = 100mm$

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 10 = 80mm$

iii) 189.46mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 220, 230:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (5 + 3.375) × 4 + (0.3 × 3.5 × 4) = 37.7KN

$$V_{AS1} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{33.64 + 69.99}{4}\right) = -13.65KN$$

$$V_{BS1} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{33.64 + 69.99}{4}\right) = 58.89KN$$

$$V_{AS2} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{33.64 + 69.99}{4}\right) = 58.89KN$$

$$V_{BS2} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{33.64 + 69.99}{4}\right) = -13.65KN$$

Shear force from analysis = 54.44 KN

$\therefore V_u = 58.89KN$

$$\tau_v = \frac{V_u}{bd} = \frac{58.89\times 10^3}{300\times 412.5} = 0.476 N/mm^2$$

$P_t = 0.19$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.312$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 58.89 \times 10^3 - 0.312 \times 300 \times 412.5$

$= 20.28 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87\times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87\times 415\times (2\times 50)\times 412.5}{20.28\times 10^3}$$

$$= 734.38 mm$$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13 mm$

ii) $8 \times \Phi_{min} = 8 \times 10 = 80$mm

iii) 734.38mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 221:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2\times\left(\frac{92.7}{2}\right) - 1.4\times\left(\frac{198.22+256.05}{4}\right) = -103.37 KN$$

$$V_{BS1} = 1.2\times\left(\frac{92.7}{2}\right) + 1.4\times\left(\frac{198.22+256.05}{4}\right) = 214.61 KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{198.22 + 256.05}{4}\right) = 214.61KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{198.22 + 256.05}{4}\right) = -103.37KN$$

Shear force from analysis = 163.26 KN

$\therefore$ V_u = 214.61KN

$$\tau_v = \frac{V_u}{bd} = \frac{214.61 \times 10^3}{300 \times 412.5} = 1.734 N/mm^2$$

P_t = 1.301

Refering table 19 of IS 456: 2000

$\therefore$ τ_c = 0.68 N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 214.61 \times 10^3 - 0.68 \times 300 \times 412.5$

$= 130.46 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{130.46 \times 10^3}$$

$$= 114.16mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 174.82mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 222:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = -86.26KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = 197.5KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = 197.5KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{167.79 + 237.57}{4}\right) = -86.26KN$$

Shear force from analysis = 150.28 KN

$\therefore V_u = 197.5$KN

$$\tau_v = \frac{V_u}{bd} = \frac{197.5 \times 10^3}{300 \times 412.5} = 1.596 N/mm^2$$

$P_t = 1.118$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.644$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 197.5 \times 10^3 - 0.644 \times 300 \times 412.5$

$= 117.81 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{117.81 \times 10^3}$$

$$= 126.42mm$$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 126.42mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 223:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = -60.28KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = 171.52KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = 171.52KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{132.92 + 198.22}{4}\right) = -60.28KN$$

Shear force from analysis = 135.28 KN

$\therefore V_u = 171.52$KN

$$\tau_v = \frac{V_u}{bd} = \frac{171.52 \times 10^3}{300 \times 412.5} = 1.386 N/mm^2$$

$P_t = 0.8847$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.592$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 171.52 \times 10^3 - 0.592 \times 300 \times 412.5$

$= 98.26 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{98.26 \times 10^3}$$

$$= 151.57mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 174.82mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 224:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (18.75 + 3.375) × 4 + (0.3 × 3.5 × 4) = 92.7KN

$$V_{AS1} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{80.71 + 150.84}{4}\right) = -25.42KN$$

$$V_{BS1} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{80.71 + 150.84}{4}\right) = 136.66KN$$

$$V_{AS2} = 1.2 \times \left(\frac{92.7}{2}\right) + 1.4 \times \left(\frac{80.71 + 150.84}{4}\right) = 136.66KN$$

$$V_{BS2} = 1.2 \times \left(\frac{92.7}{2}\right) - 1.4 \times \left(\frac{80.71 + 150.84}{4}\right) = -25.42KN$$

Shear force from analysis = 110.38 KN

$\therefore V_u$ = 136.66KN

$$\tau_v = \frac{V_u}{bd} = \frac{136.66 \times 10^3}{300 \times 412.5} = 0.892 N/mm^2$$

P_t = 0.4874

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.474$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 136.66 \times 10^3 - 0.474 \times 300 \times 412.5$

$= 51.72 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{51.72 \times 10^3}$$

$$= 287.96 mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13 mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 287.96mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 225:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (5 + 3.375) × 4 + (0.3 × 3.5 × 4) = 37.7KN

$$V_{AS1} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{32.47 + 65.8}{4}\right) = -11.77 KN$$

$$V_{BS1} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{32.47 + 65.8}{4}\right) = 57.01 KN$$

$$V_{AS2} = 1.2 \times \left(\frac{37.7}{2}\right) + 1.4 \times \left(\frac{32.47 + 65.8}{4}\right) = 57.01 KN$$

$$V_{BS2} = 1.2 \times \left(\frac{37.7}{2}\right) - 1.4 \times \left(\frac{32.47 + 65.8}{4}\right) = -11.77 KN$$

Shear force from analysis = 52.06 KN

∴ V_u = 57.01KN

$$\tau_v = \frac{V_u}{bd} = \frac{57.01 \times 10^3}{300 \times 412.5} = 0.461 N/mm^2$$

P_t = 0.1828

Refering table 19 of IS 456: 2000

∴ τ_c = 0.306 N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 57.01 \times 10^3 - 0.306 \times 300 \times 412.5$

$= 19.14 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{19.14 \times 10^3}$$

$$= 778.13mm$$

According to IS 13920:1993

$S_{v\,min} = 100mm$

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96mm$

iii) 778.13mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Design for shear reinforcement for beams of intermediate frame in YZ plane:

Beam No. 236, 256:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{266.62 + 329.5}{4}\right) = -142.79KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{266.62 + 329.5}{4}\right) = 274.5KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{266.62 + 329.5}{4}\right) = 274.5KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{266.62 + 329.5}{4}\right) = -142.79KN$$

Shear force from analysis = 202.16 KN

$\therefore V_u = 274.5KN$

$$\tau_v = \frac{V_u}{bd} = \frac{274.5 \times 10^3}{300 \times 412.5} = 2.218 N/mm^2$$

$P_t = 1.714$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.7456 \text{ N/mm}^2$

$\tau_{c\ max} = 2.8 \text{ N/mm}^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 274.5 \times 10^3 - 0.7456 \times 300 \times 412.5$

$= 182.23 \times 10^3 \text{N/mm}^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{182.23 \times 10^3}$$

$$= 81.73mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 81.73mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 237, 257:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{202.2 + 282.09}{4}\right) = -103.65KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{202.2 + 282.09}{4}\right) = 235.36KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{202.2 + 282.09}{4}\right) = 235.36KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{202.2 + 282.09}{4}\right) = -103.65KN$$

Shear force from analysis = 175.81 KN

$\therefore V_u = 235.36\text{KN}$

$$\tau_v = \frac{V_u}{bd} = \frac{235.36 \times 10^3}{300 \times 412.5} = 2.218 N/mm^2$$

$P_t = 1.325$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.685\ \text{N/mm}^2$

$\tau_{c\ max} = 2.8\ \text{N/mm}^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 235.36 \times 10^3 - 0.685 \times 300 \times 412.5$

$= 150.59 \times 10^3 \text{N/mm}^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{150.59 \times 10^3}$$

$= 98.9mm$

According to IS 13920:1993

$S_{v\ min} = 100\text{mm}$

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96\text{mm}$

iii) 98.9mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 238, 258:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = -80.95KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = 212.66KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = 212.66KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = -80.95KN$$

Shear force from analysis = 159.02 KN

$\therefore V_u = 212.66KN$

$$\tau_v = \frac{V_u}{bd} = \frac{212.66 \times 10^3}{300 \times 412.5} = 1.718 N/mm^2$$

$P_t = 1.118$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.644\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 212.66 \times 10^3 - 0.644 \times 300 \times 412.5$

$= 132.97 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{132.97 \times 10^3}$$

$$= 112.01mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 112.01mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 239, 259:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{103.37 + 179.79}{4}\right) = -33.25KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{103.37 + 179.79}{4}\right) = 164.96KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{103.37 + 179.79}{4}\right) = 164.96KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{103.37 + 179.79}{4}\right) = -33.25KN$$

Shear force from analysis = 127.49 KN

∴ V_u = 164.96KN

$$\tau_v = \frac{V_u}{bd} = \frac{164.96 \times 10^3}{300 \times 412.5} = 1.333 N/mm^2$$

P_t = 0.649

Refering table 19 of IS 456: 2000

∴ $\tau_c = 0.528$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 164.96 \times 10^3 - 0.528 \times 300 \times 412.5$

$= 99.62 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{99.62 \times 10^3}$$

$$= 149.5mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 149.5mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 240, 260:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (9.69 + 3.375) × 4 + (0.3 × 6.78 × 4) = 60.4KN

$$V_{AS1} = 1.2 \times \left(\frac{60.4}{2}\right) - 1.4 \times \left(\frac{43.03 + 75.34}{4}\right) = -5.19KN$$

$$V_{BS1} = 1.2 \times \left(\frac{60.4}{2}\right) + 1.4 \times \left(\frac{43.03 + 75.34}{4}\right) = 77.67KN$$

$$V_{AS2} = 1.2 \times \left(\frac{60.4}{2}\right) + 1.4 \times \left(\frac{43.03 + 75.34}{4}\right) = 77.67KN$$

$$V_{BS2} = 1.2 \times \left(\frac{60.4}{2}\right) - 1.4 \times \left(\frac{43.03 + 75.34}{4}\right) = -5.19KN$$

Shear force from analysis = 59.7 KN

$\therefore V_u$ = 77.67KN

$$\tau_v = \frac{V_u}{bd} = \frac{77.67 \times 10^3}{300 \times 412.5} = 0.628 N/mm^2$$

P_t = 0.246

Refering table 19 of IS 456: 2000

$\therefore \tau_c$ = 0.357 N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 77.67 \times 10^3 - 0.357 \times 300 \times 412.5$

$= 33.49 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{33.49 \times 10^3}$$

$$= 444.71mm$$

According to IS 13920:1993

$S_{v\,min} = 100mm$

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 10 = 80mm$

iii) 444.71mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 241, 251:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = -96.85KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = 228.55KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = 228.55KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{198.22 + 266.62}{4}\right) = -96.85KN$$

Shear force from analysis = 175.76 KN

$\therefore V_u = 228.55KN$

$$\tau_v = \frac{V_u}{bd} = \frac{228.55 \times 10^3}{300 \times 412.5} = 1.847 N/mm^2$$

$P_t = 1.301$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.68$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 228.55 \times 10^3 - 0.68 \times 300 \times 412.5$

$= 144.4 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{144.4 \times 10^3}$$

$$= 103.14 mm$$

According to IS 13920:1993

$S_{v\ min} = 100$mm

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13 mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 103.14mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 242, 252:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = -80.95 KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = 212.66 KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = 212.66KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = -80.95KN$$

Shear force from analysis = 165.36 KN

$\therefore V_u = 212.66KN$

$$\tau_v = \frac{V_u}{bd} = \frac{212.66 \times 10^3}{300 \times 412.5} = 1.718 N/mm^2$$

$P_t = 1.118$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.644\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 212.66 \times 10^3 - 0.644 \times 300 \times 412.5$

$= 132.97 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{132.97 \times 10^3}$$

$$= 112mm$$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128mm$

iii) 112mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 243, 253:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{140.88 + 213.88}{4}\right) = -58.31KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{140.88 + 213.88}{4}\right) = 190.02KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{140.88 + 213.88}{4}\right) = 190.02KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{140.88 + 213.88}{4}\right) = -58.31KN$$

Shear force from analysis = 149.15 KN

$\therefore V_u = 190.02KN$

$$\tau_v = \frac{V_u}{bd} = \frac{190.02 \times 10^3}{300 \times 412.5} = 1.536 N/mm^2$$

$P_t = 0.995$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.609\ N/mm^2$

$\tau_{c\ max} = 2.8\ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 192.02 \times 10^3 - 0.609 \times 300 \times 412.5$

$= 114.66 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{114.66 \times 10^3}$$

$= 129.89mm$

According to IS 13920:1993

$S_{v\ min} = 100mm$

$S_{v\ max} =$ i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128mm$

iii) 129.89mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 244, 254:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{92.7 + 167.79}{4}\right) = -25.32KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{92.7 + 167.79}{4}\right) = 157.03KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{92.7 + 167.79}{4}\right) = 157.03KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{92.7 + 167.79}{4}\right) = -25.32KN$$

Shear force from analysis = 123.99 KN

$\therefore V_u = 157.03KN$

$$\tau_v = \frac{V_u}{bd} = \frac{157.03 \times 10^3}{300 \times 412.5} = 1.269 N/mm^2$$

$P_t = 0.571$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.503 \ N/mm^2$

$\tau_{c\ max} = 2.8 \ N/mm^2$ for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 157.03 \times 10^3 - 0.503 \times 300 \times 412.5$

$= 94.78 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{94.78 \times 10^3}$$

$$= 157.14mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 10 = 80$mm

iii) 157.14mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 245, 255:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (9.69 + 3.375) × 4 + (0.3 × 6.78 × 4) = 60.4KN

$$V_{AS1} = 1.2 \times \left(\frac{60.4}{2}\right) - 1.4 \times \left(\frac{43.03 + 69.99}{4}\right) = -3.32KN$$

$$V_{BS1} = 1.2 \times \left(\frac{60.4}{2}\right) + 1.4 \times \left(\frac{43.03 + 69.99}{4}\right) = 75.78KN$$

$$V_{AS2} = 1.2 \times \left(\frac{60.4}{2}\right) + 1.4 \times \left(\frac{43.03 + 69.99}{4}\right) = 75.78KN$$

$$V_{BS2} = 1.2 \times \left(\frac{60.4}{2}\right) - 1.4 \times \left(\frac{43.03 + 69.99}{4}\right) = -3.32KN$$

Shear force from analysis = 60.17 KN

$\therefore V_u$ = 75.78KN

$$\tau_v = \frac{V_u}{bd} = \frac{75.78 \times 10^3}{300 \times 412.5} = 0.612N/mm^2$$

P_t = 0.246

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.357$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 75.78 \times 10^3 - 0.357 \times 300 \times 412.5$

$= 31.6 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{31.6 \times 10^3}$$

$$= 471.3mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 10 = 80$mm

iii) 471.3mm

Provide 8Φ two legged stirrup @ 80mm c/c.

Beam No. 246:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{198.22 + 282.09}{4}\right) = -102.25KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{198.22 + 282.09}{4}\right) = 233.96KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{198.22 + 282.09}{4}\right) = 233.96KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{198.22 + 282.09}{4}\right) = -102.25KN$$

Shear force from analysis = 178.2 KN

$\therefore V_u$ = 233.96KN

$$\tau_v = \frac{V_u}{bd} = \frac{233.96 \times 10^3}{300 \times 412.5} = 1.89N/mm^2$$

P_t = 1.301

Refering table 19 of IS 456: 2000

$\therefore \tau_c$ = 0.68 N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 233.96 \times 10^3 - 0.68 \times 300 \times 412.5$

$= 149.81 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{149.81 \times 10^3}$$

$$= 99.41 mm$$

According to IS 13920:1993

$S_{v\,min}$ = 100mm

$S_{v\,max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13 mm$

ii) $8 \times \Phi_{min} = 8 \times 20 = 160$mm

iii) 99.41mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 247:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = -80.95 KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = 212.66 KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = 212.66 KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{167.79 + 251.66}{4}\right) = -80.95 KN$$

Shear force from analysis = 165.36 KN

$\therefore V_u = 212.66$KN

$$\tau_v = \frac{V_u}{bd} = \frac{212.66 \times 10^3}{300 \times 412.5} = 1.718 N/mm^2$$

P_t = 1.118

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.644$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 212.66 \times 10^3 - 0.644 \times 300 \times 412.5$

$= 132.97 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{132.97 \times 10^3}$$

$$= 112mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 112mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 248:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{132.93 + 213.88}{4}\right) = -55.53KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{132.93 + 213.88}{4}\right) = 187.24KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{132.93 + 213.88}{4}\right) = 187.24KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{132.93 + 213.88}{4}\right) = -55.53KN$$

Shear force from analysis = 148.28 KN

$\therefore$ V_u = 187.24KN

$$\tau_v = \frac{V_u}{bd} = \frac{187.24 \times 10^3}{300 \times 412.5} = 1.513 N/mm^2$$

P_t = 0.8847

Refering table 19 of IS 456: 2000

$\therefore$ τ_c = 0.592 N/mm^2

$\tau_{c\ max}$ = 2.8 N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 187.24 \times 10^3 - 0.592 \times 300 \times 412.5$

$= 113.98 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{113.98 \times 10^3}$$

$$= 130.66mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 130.66mm

Provide 8Φ two legged stirrup @ 95mm c/c.

Beam No. 249:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (22.03 + 3.375) × 4 + (0.3 × 6.78 × 4) = 109.76KN

$$V_{AS1} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{80.71 + 167.79}{4}\right) = -21.12KN$$

$$V_{BS1} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{80.71 + 167.79}{4}\right) = 152.83KN$$

$$V_{AS2} = 1.2 \times \left(\frac{109.76}{2}\right) + 1.4 \times \left(\frac{80.71 + 167.79}{4}\right) = 152.83KN$$

$$V_{BS2} = 1.2 \times \left(\frac{109.76}{2}\right) - 1.4 \times \left(\frac{80.71 + 167.79}{4}\right) = -21.12KN$$

Shear force from analysis = 122.29 KN

$\therefore V_u$ = 152.83KN

$$\tau_v = \frac{V_u}{bd} = \frac{152.83 \times 10^3}{300 \times 412.5} = 1.235N/mm^2$$

P_t = 0.4874

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.474$ N/mm^2

$\tau_{c\ max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\ max}$

$V_{us} = V_u - \tau_c bd$

$= 152.83 \times 10^3 - 0.474 \times 300 \times 412.5$

$= 94.17 \times 10^3$N/mm^2

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{94.17 \times 10^3}$$

$$= 158.15mm$$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 16 = 128$mm

iii) 158.15mm

Provide 8Φ two legged stirrup @ 100mm c/c.

Beam No. 250:

Self wt of beam = 0.3 × 0.45 × 25 = 3.375KN/m

WL = (9.69 + 3.375) × 4 + (0.3 × 6.78 × 4) = 60.4KN

$$V_{AS1} = 1.2 \times \left(\frac{60.4}{2}\right) - 1.4 \times \left(\frac{32.52 + 65.8}{4}\right) = 1.828KN$$

$$V_{BS1} = 1.2 \times \left(\frac{60.4}{2}\right) + 1.4 \times \left(\frac{32.52 + 65.8}{4}\right) = 70.65KN$$

$$V_{AS2} = 1.2 \times \left(\frac{60.4}{2}\right) + 1.4 \times \left(\frac{32.52 + 65.8}{4}\right) = 70.65KN$$

$$V_{BS2} = 1.2 \times \left(\frac{60.4}{2}\right) - 1.4 \times \left(\frac{32.52 + 65.8}{4}\right) = 1.828KN$$

Shear force from analysis = 58.01 KN

$\therefore V_u = 70.65KN$

$$\tau_v = \frac{V_u}{bd} = \frac{70.65 \times 10^3}{300 \times 412.5} = 0.571N/mm^2$$

$P_t = 0.183$

Refering table 19 of IS 456: 2000

$\therefore \tau_c = 0.306$ N/mm^2

$\tau_{c\,max} = 2.8$ N/mm^2 for M20 concrete.

$\tau_v > \tau_c < \tau_{c\,max}$

$V_{us} = V_u - \tau_c bd$

$= 70.65 \times 10^3 - 0.306 \times 300 \times 412.5$

$= 32.78 \times 10^3 N/mm^2$

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{32.78 \times 10^3}$$

$= 454.34mm$

According to IS 13920:1993

$S_{v\ min}$ = 100mm

$S_{v\ max}$ = i) $\frac{d}{4} = \frac{412.5}{4} = 103.13mm$

ii) $8 \times \Phi_{min} = 8 \times 12 = 96$mm

iii) 454.34mm

Provide 8Φ two legged stirrup @ 95mm c/c.

<u>Shear reinforcement and special confining reinforcement for columns of end frame in XZ plane:</u>

<u>Column No. 5, 95:</u>

Shear reinforcement:

Capacity based shear = 108.04KN.

Shear force from analysis = 43.94KN

$\therefore V_u$ = 108.04KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$= 1 + \frac{3 \times 126.12 \times 10^3}{400 \times 500 \times 20}$

= 1.095

$\therefore \tau_c = 0.82 \times 1.095 = 0.898 N/mm^2$

$V_c = \tau_c \times bd$

$= 0.898 \times 400 \times 450$

$= 161.64 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$

$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$

= 225.67mm.

Maximum spacing of stirrups:

i) $0.75 \times d = 0.75 \times 450 = 337.5$mm

ii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

iii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

i) Depth of the member = 500mm

ii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

iii) 450mm.

∴ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1\right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

$= 340 \times 440\ mm^2$ (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1\right)$$

$= 99.36 mm^2$

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 35, 65:</u>

Shear reinforcement:

Capacity based shear = 170.67KN.

Shear force from analysis = 61.7KN

$\therefore V_u = 170.67KN$

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 143.42 \times 10^3}{400 \times 550 \times 20}$$

$= 1.098$

$\therefore \tau_c = 0.82 \times 1.098 = 0.9 N/mm^2$

$V_c = \tau_c \times bd$

$= 0.9 \times 400 \times 500$

$= 180 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

iv) $0.75 \times d = 0.75 \times 500 = 375$mm

v) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

vi) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

iv) Depth of the member = 550mm

v) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

vi) 450mm.

$\therefore L_0 = 550$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

$= 340 \times 490$ mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

$= 94.53$mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{94.53} = 83.07mm$$

Provide 10Φ hook @ 80mm c/c.

<u>Column No. 4, 94:</u>

Shear reinforcement:

Capacity based shear = 134.74KN.

Shear force from analysis = 68.15KN

$\therefore V_u = 134.74$KN

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 342.42 \times 10^3}{400 \times 500 \times 20}$$

$= 1.257$

$\therefore \tau_c = 0.82 \times 1.257 = 1.03$N/mm^2

$V_c = \tau_c \times bd$

$= 1.03 \times 400 \times 450$

$= 185.4 \times 10^3$N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

vii) $0.75 \times d = 0.75 \times 450 = 337.5mm$

viii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

ix) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

vii) Depth of the member = 500mm

viii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

ix) 450mm.

∴ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

$= 340 \times 440\ mm^2$ (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1\right)$$

$= 99.36 mm^2$

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 34, 64:</u>

Shear reinforcement:

Capacity based shear = 240.6KN.

Shear force from analysis = 112.34KN

$\therefore V_u = 240.6KN$

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 383.26 \times 10^3}{400 \times 550 \times 20}$$

$= 1.261$

$\therefore \tau_c = 0.82 \times 1.261 = 1.034 N/mm^2$

$V_c = \tau_c \times bd$

$= 1.034 \times 400 \times 500$

$= 260.8 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67mm.$

Maximum spacing of stirrups:

x) $0.75 \times d = 0.75 \times 500 = 375$mm

xi) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xii) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

x) Depth of the member = 550mm

xi) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xii) 450mm.

$\therefore L_0 = 550$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

$= 94.53\text{mm}^2$

Use 10Φ hook.

Spacing = $\dfrac{100 \times 78.53}{94.53} = 83.07mm$

Provide 10Φ hook @ 80mm c/c.

<u>Column No. 3, 93:</u>

Shear reinforcement:

Capacity based shear = 168.17KN.

Shear force from analysis = 81.89KN

$\therefore V_u = 168.17\text{KN}$

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$= 1 + \dfrac{3 \times 589.27 \times 10^3}{400 \times 500 \times 20}$

$= 1.442$

$\therefore \tau_c = 0.82 \times 1.442 = 1.182\text{N/mm}^2$

$V_c = \tau_c \times bd$

$= 1.182 \times 400 \times 450$

$= 212.76 \times 10^3\text{N}$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$\therefore S_{v\min} = \dfrac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$

$= \dfrac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$

$= 225.67\text{mm.}$

Maximum spacing of stirrups:

xiii) $0.75 \times d = 0.75 \times 450 = 337.5$mm

xiv) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xv) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xiii) Depth of the member = 500mm

xiv) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xv) 450mm.

∴ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1\right)$$

$= 99.36\text{mm}^2$

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 33, 63:</u>

Shear reinforcement:

Capacity based shear = 304.14KN.

Shear force from analysis = 143.84KN

$\therefore V_u = 304.14\text{KN}$

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 622.68 \times 10^3}{400 \times 550 \times 20}$$

$= 1.425$

$\therefore \tau_c = 0.82 \times 1.425 = 1.1685\text{N/mm}^2$

$V_c = \tau_c \times bd$

$= 1.1685 \times 400 \times 500$

$= 233.7 \times 10^3\text{N}$

$V_u > V_c$

$V_{us} = 304.14 - 233.7 = 70.44$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{70.44 \times 10^3}$$

$= 256.28mm$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

xvi) 0.75 × d = 0.75 × 500 = 375mm

xvii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xviii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xvi) Depth of the member = 550mm

xvii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xviii) 450mm.

∴ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{94.53} = 83.07mm$$

Provide 10Φ hook @ 80mm c/c.

Column No. 2, 92:

Shear reinforcement:

Capacity based shear = 171.82KN.

Shear force from analysis = 83.19KN

$\therefore V_u$ = 171.82KN

τ_c = 0.82 × δ

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 852.14 \times 10^3}{400 \times 500 \times 20}$$

= 1.639

$\therefore \tau_c$ = 0.82 × 1.639 = 1.344N/mm^2

$V_c = \tau_c \times bd$

= 1.344 × 400 × 450

= 241.92 × 10^3N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

xix) 0.75 × d = 0.75 × 450 = 337.5mm

xx) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xxi) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xix) Depth of the member = 500mm

xx) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xxi) 450mm.

$\therefore L_0 = 508\text{mm} \approx 510$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

$= 400 - 40 - 40 + 10 + 10 = 340$mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

$= 340 \times 440$ mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

$= 99.36$mm^2

Use 10Φ hook.

Spacing $= \dfrac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

Column No. 32, 62:

Shear reinforcement:

Capacity based shear = 344.1KN.

Shear force from analysis = 167.98KN

$\therefore V_u = 344.1$KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 885.6 \times 10^3}{400 \times 550 \times 20}$$

$= 1.604$

$\therefore \tau_c = 0.82 \times 1.604 = 1.315$N/mm^2

$V_c = \tau_c \times bd$

$= 1.315 \times 400 \times 500$

$= 263 \times 10^3$N

$V_u > V_c$

$V_{us} = 344.1 - 261 = 81.1$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{81.1 \times 10^3}$$

$$= 222.6mm$$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

xxii) 0.75 × d = 0.75 × 500 = 375mm

xxiii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xxiv) 222.6mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xxii) Depth of the member = 550mm

xxiii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xxiv) 450mm.

∴ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$= 0.18 \times S \times h \times \dfrac{f_{ck}}{f_y} \times \left(\dfrac{A_g}{A_k} - 1 \right)$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \dfrac{20}{415} \times \left(\dfrac{400 \times 550}{340 \times 490} - 1 \right)$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\dfrac{100 \times 78.53}{94.53} = 83.07 mm$

Provide 10Φ hook @ 80mm c/c.

<u>Column No. 1, 91:</u>

Shear reinforcement:

Capacity based shear = 177.41KN.

Shear force from analysis = 83.19KN

$\therefore V_u$ = 177.41KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$= 1 + \dfrac{3 \times 1131.58 \times 10^3}{400 \times 500 \times 20}$

= 1.85

$\therefore \tau_c = 0.82 \times 1.85 = 1.517 N/mm^2$

$V_c = \tau_c \times bd$

= 1.517 × 400 × 450

$= 273.06 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67$mm.

Maximum spacing of stirrups:

xxv) $0.75 \times d = 0.75 \times 450 = 337.5$mm

xxvi) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

xxvii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xxv) Depth of the member = 500mm

xxvi) $\frac{1}{6}$ (Clear span) $= \frac{1}{6} \times (4.5 - 0.45) = 0.675m = 675mm$

xxvii) 450mm.

$\therefore L_0 = 675$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) $= \frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$

= $99.36mm^2$

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 31, 61:</u>

Shear reinforcement:

Capacity based shear = 291.3KN.

Shear force from analysis = 147.63KN

$\therefore V_u$ = 291.3KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$= 1 + \frac{3 \times 1174.85 \times 10^3}{400 \times 550 \times 20}$

= 1.801

$\therefore \tau_c = 0.82 \times 1.801 = 1.477 N/mm^2$

$V_c = \tau_c \times bd$

= 1.477 × 400 × 500

$= 295.4 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67mm.$

Maximum spacing of stirrups:

xxviii) $0.75 \times d = 0.75 \times 500 = 375mm$

xxix) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200mm$

xxx) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xxviii) Depth of the member = 550mm

xxix) $\frac{1}{6}$ (Clear span) $= \frac{1}{6} \times (4.5 - 0.45) = 0.675m = 675mm$

xxx) 450mm.

$\therefore L_0 = 675$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) $= \frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{94.53} = 83.07 mm$$

Provide 10Φ hook @ 80mm c/c.

<u>Shear reinforcement and special confining reinforcement for columns of frame next to end frame in XZ plane:</u>

<u>Column No. 10, 100:</u>

Shear reinforcement:

Capacity based shear = 126.61KN.

Shear force from analysis = 43.53KN

∴ V_u = 126.61KN

τ_c = 0.82 × δ

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 167.14 \times 10^3}{400 \times 500 \times 20}$$

= 1.125

∴ τ_c = 0.82 × 1.125 = 0.923N/mm^2

$V_c = \tau_c \times bd$

$= 0.923 \times 400 \times 450$

$= 184.6 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

xxxi) 0.75 × d = 0.75 × 450 = 337.5mm

xxxii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xxxiii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xxxi) Depth of the member = 500mm

xxxii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xxxiii) 450mm.

$\therefore L_0 = 508mm \approx 510$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

Column No. 40, 70:

Shear reinforcement:

Capacity based shear = 168.99KN.

Shear force from analysis = 60.8KN

∴ V_u = 168.99KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 214.26 \times 10^3}{400 \times 550 \times 20}$$

= 1.146

∴ $\tau_c = 0.82 \times 1.146 = 0.94$N/mm^2

$V_c = \tau_c \times bd$

$= 0.94 \times 400 \times 500$

$= 188 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67$mm.

Maximum spacing of stirrups:

xxxiv) $0.75 \times d = 0.75 \times 500 = 375$mm

xxxv) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

xxxvi) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xxxiv) Depth of the member = 550mm

xxxv) $\frac{1}{6}$ (Clear span) $= \frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xxxvi) 450mm.

$\therefore L_0 = 550$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) $= \frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{94.53} = 83.07mm$

Provide 10Φ hook @ 80mm c/c.

Column No. 9, 99:

Shear reinforcement:

Capacity based shear = 187.1KN.

Shear force from analysis = 81.71KN

∴ V_u = 187.1KN

τ_c = 0.82 × δ

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 424.32 \times 10^3}{400 \times 500 \times 20}$$

= 1.318

∴ τ_c = 0.82 × 1.318 = 1.08N/mm^2

$V_c = \tau_c \times bd$

$= 1.08 \times 400 \times 450$

$= 194.4 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67$mm.

Maximum spacing of stirrups:

xxxvii) $0.75 \times d = 0.75 \times 450 = 337.5$mm

xxxviii) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

xxxix) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xxxvii) Depth of the member = 500mm

xxxviii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xxxix) 450mm.

$\therefore L_0 = 508\text{mm} \approx 510$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

Column No. 39, 69:

Shear reinforcement:

Capacity based shear = 247.7KN.

Shear force from analysis = 114.05KN

∴ V_u = 247.7KN

τ_c = 0.82 × δ

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 564.89 \times 10^3}{400 \times 550 \times 20}$$

= 1.385

∴ τ_c = 0.82 × 1.385 = 1.136N/mm^2

$V_c = \tau_c \times bd$

$= 1.136 \times 400 \times 500$

$= 227.2 \times 10^3 N$

$V_u > V_c$

$V_{us} = 247.7 - 227.2 = 20.5$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{20.5 \times 10^3}$$

$= 880.61 mm$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67$mm.

Maximum spacing of stirrups:

xl) $0.75 \times d = 0.75 \times 500 = 375$mm

xli) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

xlii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xl) Depth of the member = 550mm

xli) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xlii) 450mm.

$\therefore L_0 = 550$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4}\times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18\times S\times h\times \frac{f_{ck}}{f_y}\times\left(\frac{A_g}{A_k}-1\right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18\times 100\times 340\times\frac{20}{415}\times\left(\frac{400\times 550}{340\times 490}-1\right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\frac{100\times 78.53}{94.53} = 83.07mm$

Provide 10Φ hook @ 80mm c/c.

Column No. 8, 98:

Shear reinforcement:

Capacity based shear = 230.42KN.

Shear force from analysis = 103.2KN

∴ V_u = 230.42KN

τ_c = 0.82 × δ

Where $\delta = 1+\frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 681.15 \times 10^3}{400 \times 500 \times 20}$$

$$= 1.51$$

$\therefore$ τ_c = 0.82 × 1.51 = 1.238N/mm^2

$V_c = \tau_c \times bd$

= 1.238 × 400 × 450

= 222.84 × 10^3N

$V_u > V_c$

V_{us} = 230.42 – 222.84 = 7.58 KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{7.58 \times 10^3}$$

$$= 1897.23mm$$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

xliii) 0.75 × d = 0.75 × 450 = 337.5mm

xliv) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

xlv) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xliii) Depth of the member = 500mm

xliv) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xlv) 450mm.

$\therefore$ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 38, 68:</u>

Shear reinforcement:

Capacity based shear = 304.54KN.

Shear force from analysis = 145.09KN

$\therefore V_u = 304.54KN$

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 918.01 \times 10^3}{400 \times 550 \times 20}$$

$$= 1.626$$

$\therefore \tau_c = 0.82 \times 1.626 = 1.333 N/mm^2$

$V_c = \tau_c \times bd$

$= 1.333 \times 400 \times 500$

$= 266.6 \times 10^3 N$

$V_u > V_c$

$V_{us} = 304.54 - 266.6 = 37.94$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{37.94 \times 10^3}$$

$= 475.82mm$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67$mm.

Maximum spacing of stirrups:

xlvi) $0.75 \times d = 0.75 \times 500 = 375$mm

xlvii) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

xlviii) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xlvi) Depth of the member = 550mm

xlvii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

xlviii) 450mm.

$\therefore$ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\dfrac{100 \times 78.53}{94.53} = 83.07mm$

Provide 10Φ hook @ 80mm c/c.

<u>Column No. 7, 97:</u>

Shear reinforcement:

Capacity based shear = 258.18KN.

Shear force from analysis = 117.13KN

$\therefore$ V_u = 258.18KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 937.65 \times 10^3}{400 \times 500 \times 20}$$

$$= 1.703$$

$\therefore$ $\tau_c = 0.82 \times 1.703 = 1.396 N/mm^2$

$V_c = \tau_c \times bd$

$= 1.396 \times 400 \times 450$

$= 251.28 \times 10^3 N$

$V_u > V_c$

$V_{us} = 258.18 - 251.28 = 6.9$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{6.9 \times 10^3}$$

$$= 2354.67mm$$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

xlix) $0.75 \times d = 0.75 \times 450 = 337.5$mm

l) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

li) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

xlix) Depth of the member = 500mm

l) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

li) 450mm.

∴ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

$= 340 \times 440\ mm^2$ (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1\right)$$

$= 99.36 mm^2$

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 37, 67:</u>

Shear reinforcement:

Capacity based shear = 345.59KN.

Shear force from analysis = 169.76KN

$\therefore V_u = 345.59KN$

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 1274.06 \times 10^3}{400 \times 550 \times 20}$$

$= 1.869$

$\therefore\ \tau_c = 0.82 \times 1.869 = 1.533 N/mm^2$

$V_c = \tau_c \times bd$

$= 1.533 \times 400 \times 500$

$= 306.6 \times 10^3 N$

$V_u > V_c$

$V_{us} = 345.59 - 306.6 = 38.99$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{38.99 \times 10^3}$$

$= 463mm$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lii) 0.75 × d = 0.75 × 500 = 375mm

liii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

liv) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lii) Depth of the member = 550mm

liii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

liv) 450mm.

∴ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it’s outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1\right)$$

= 94.53mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{94.53} = 83.07mm$$

Provide 10Φ hook @ 80mm c/c.

<u>Column No. 6, 96:</u>

Shear reinforcement:

Capacity based shear = 239.97KN.

Shear force from analysis = 97.32KN

∴ V_u = 239.97KN

τ_c = 0.82 × δ

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 1199.79 \times 10^3}{400 \times 500 \times 20}$$

= 1.9

∴ τ_c = 0.82 × 1.9 = 1.558N/mm^2

$V_c = \tau_c \times bd$

= 1.558 × 400 × 450

= 280.44 × 10^3N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lv) $0.75 \times d = 0.75 \times 450 = 337.5$mm

lvi) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lvii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lv) Depth of the member = 500mm

lvi) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (4.5 - 0.45) = 0.675m = 675mm$

lvii) 450mm.

∴ L_0 = 675 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 36, 66:</u>

Shear reinforcement:

Capacity based shear = 289.03KN.

Shear force from analysis = 148.58KN

$\therefore V_u$ = 289.03KN

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 1645.35 \times 10^3}{400 \times 550 \times 20}$$

= 2.122

$\therefore \tau_c = 0.82 \times 2.122 = 1.74$N/mm^2

$V_c = \tau_c \times bd$

= 1.74 × 400 × 500

= 348 × 10^{3}N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lviii) 0.75 × d = 0.75 × 500 = 375mm

lix) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lx) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lviii) Depth of the member = 550mm

lix) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (4.5 - 0.45) = 0.675m = 675mm$

lx) 450mm.

∴ L_0 = 675 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{94.53} = 83.07mm$

Provide 10Φ hook @ 80mm c/c.

<u>Shear reinforcement and special confining reinforcement for columns of intermediate frame in XZ plane:</u>

<u>Column No. 15, 105:</u>

Shear reinforcement:

Capacity based shear = 115.52KN.

Shear force from analysis = 43.53KN

∴ V_u = 115.52KN

τ_c = 0.82 × δ

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$= 1 + \frac{3 \times 167.14 \times 10^3}{400 \times 500 \times 20}$

= 1.125

∴ τ_c = 0.82 × 1.125 = 0.923N/mm^2

$V_c = \tau_c \times bd$

= 0.923 × 400 × 450

= 184.6 × 10^3N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lxi) 0.75 × d = 0.75 × 450 = 337.5mm

lxii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lxiii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxi) Depth of the member = 500mm

lxii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxiii) 450mm.

∴ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

Column No. 45, 75:

Shear reinforcement:

Capacity based shear = 168.99KN.

Shear force from analysis = 60.8KN

∴ V_u = 168.99KN

τ_c = 0.82 × δ

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 214.26 \times 10^3}{400 \times 550 \times 20}$$

= 1.146

∴ τ_c = 0.82 × 1.146 = 0.94N/mm^2

$V_c = \tau_c \times bd$

= 0.94 × 400 × 500

= 188 × 10^{3}N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lxiv) 0.75 × d = 0.75 × 500 = 375mm

lxv) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lxvi) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxiv) Depth of the member = 550mm

lxv) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxvi) 450mm.

∴ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{94.53} = 83.07mm$$

Provide 10Φ hook @ 80mm c/c.

Column No. 14, 104:

Shear reinforcement:

Capacity based shear = 171.26KN.

Shear force from analysis = 81.71KN

∴ V_u = 171.26KN

τ_c = 0.82 × δ

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 431.05 \times 10^3}{400 \times 500 \times 20}$$

= 1.32

∴ τ_c = 0.82 × 1.32 = 1.082N/mm^2

$V_c = \tau_c \times bd$

= 1.082 × 400 × 450

= 194.76 × 10^3N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lxvii) 0.75 × d = 0.75 × 450 = 337.5mm

lxviii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lxix) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxvii) Depth of the member = 500mm

lxviii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxix) 450mm.

∴ L_0 = 508mm ≈ 510 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1\right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

$$= 99.36 \text{mm}^2$$

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{99.36} = 79.03mm$$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 44, 74:</u>

Shear reinforcement:

Capacity based shear = 247.7KN.

Shear force from analysis = 114.05KN

$\therefore V_u = 247.7$KN

$\tau_c = 0.82 \times \delta$

$$\text{Where } \delta = 1 + \frac{3P_u}{A_g f_{ck}}$$

$$= 1 + \frac{3 \times 564.89 \times 10^3}{400 \times 550 \times 20}$$

$$= 1.385$$

$\therefore \tau_c = 0.82 \times 1.385 = 1.136$N/mm^2

$V_c = \tau_c \times bd$

$= 1.136 \times 400 \times 500$

$= 227.2 \times 10^3$N

$V_u > V_c$

$V_{us} = 247.7 – 227.2 = 20.5$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{20.5 \times 10^3}$$

$$= 880.61 mm$$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lxx) 0.75 × d = 0.75 × 500 = 375mm

lxxi) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lxxii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxx) Depth of the member = 550mm

lxxi) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxxii) 450mm.

∴ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

$$\text{Spacing} = \frac{100 \times 78.53}{94.53} = 83.07mm$$

Provide 10Φ hook @ 80mm c/c.

Column No. 13, 103:

Shear reinforcement:

Capacity based shear = 215.06KN.

Shear force from analysis = 103.2KN

∴ V_u = 215.06KN

τ_c = 0.82 × δ

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 690.59 \times 10^3}{400 \times 500 \times 20}$$

= 1.52

∴ τ_c = 0.82 × 1.52 = 1.246N/mm^2

$V_c = \tau_c \times bd$

$= 1.246 \times 400 \times 450$

$= 224.28 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lxxiii) $0.75 \times d = 0.75 \times 450 = 337.5$mm

lxxiv) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lxxv) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxxiii) Depth of the member = 500mm

lxxiv) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxxv) 450mm.

$\therefore L_0 = 508mm \approx 510$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

∴ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 43, 73:</u>

Shear reinforcement:

Capacity based shear = 304.54KN.

Shear force from analysis = 145.09KN

∴ V_u = 304.54KN

τ_c = 0.82 × δ

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 918.01 \times 10^3}{400 \times 550 \times 20}$$

= 1.626

∴ τ_c = 0.82 × 1.626 = 1.333N/mm^2

$V_c = \tau_c \times bd$

$= 1.333 \times 400 \times 500$

$= 266.6 \times 10^3 N$

$V_u > V_c$

$V_{us} = 304.54 - 266.6 = 37.94$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{37.94 \times 10^3}$$

$= 475.82mm$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

= 225.67mm.

Maximum spacing of stirrups:

lxxvi) 0.75 × d = 0.75 × 500 = 375mm

lxxvii) (1/2) × Least lateral dimension = (1/2) × 400 = 200mm

lxxviii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxxvi) Depth of the member = 550mm

lxxvii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxxviii) 450mm.

$\therefore L_0 = 550$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 550}{340 \times 490} - 1 \right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{94.53} = 83.07mm$

Provide 10Φ hook @ 80mm c/c.

<u>Column No. 12, 102:</u>

Shear reinforcement:

Capacity based shear = 238.95KN.

Shear force from analysis = 117.13KN

$\therefore V_u$ = 238.95KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 948.65 \times 10^3}{400 \times 500 \times 20}$$

$$= 1.71$$

$\therefore\ \tau_c = 0.82 \times 1.71 = 1.4 N/mm^2$

$V_c = \tau_c \times bd$

$= 1.4 \times 400 \times 450$

$= 252 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67mm.$

Maximum spacing of stirrups:

lxxix) $0.75 \times d = 0.75 \times 450 = 337.5mm$

lxxx) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200mm$

lxxxi) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxxix) Depth of the member = 500mm

lxxx) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (3.5 - 0.45) = 0.508m = 508mm$

lxxxi) 450mm.

$\therefore L_0 = 508mm \approx 510$ mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 42, 72:</u>

Shear reinforcement:

Capacity based shear = 345.59KN.

Shear force from analysis = 169.76KN

$\therefore V_u$ = 345.59KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \frac{3P_u}{A_g f_{ck}}$

$$= 1 + \frac{3 \times 1274.06 \times 10^3}{400 \times 550 \times 20}$$

$= 1.869$

$\therefore\ \tau_c = 0.82 \times 1.869 = 1.533 \text{N/mm}^2$

$V_c = \tau_c \times bd$

$= 1.533 \times 400 \times 500$

$= 306.6 \times 10^3 \text{N}$

$V_u > V_c$

$V_{us} = 345.59 - 306.6 = 38.99$ KN.

Use 8Φ two legged stirrup.

$$S_v = \frac{0.87 \times f_y \times A_{SV} \times d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times 50) \times 412.5}{38.99 \times 10^3}$$

$= 463mm$

From minimum shear reinforcement

$$\therefore S_{v\min} = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$$

$= 225.67$mm.

Maximum spacing of stirrups:

lxxxii) $0.75 \times d = 0.75 \times 500 = 375$mm

lxxxiii) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

lxxxiv) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxxxii) Depth of the member = 550mm

lxxxiii) $\frac{1}{6}$ (Clear span) = $\frac{1}{6}\times(3.5-0.45)=0.508m=508mm$

lxxxiv) 450mm.

$\therefore$ L_0 = 550 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4}\times 400=100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18\times S\times h\times\frac{f_{ck}}{f_y}\times\left(\frac{A_g}{A_k}-1\right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18\times 100\times 340\times\frac{20}{415}\times\left(\frac{400\times 550}{340\times 490}-1\right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\frac{100\times 78.53}{94.53}=83.07mm$

Provide 10Φ hook @ 80mm c/c.

Column No. 11, 101:

Shear reinforcement:

Capacity based shear = 277.86KN.

Shear force from analysis = 97.32KN

$\therefore V_u = 277.86KN$

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$= 1 + \dfrac{3 \times 1207.59 \times 10^3}{400 \times 500 \times 20}$

$= 1.91$

$\therefore \tau_c = 0.82 \times 1.91 = 1.566 N/mm^2$

$V_c = \tau_c \times bd$

$= 1.566 \times 400 \times 450$

$= 281.88 \times 10^3 N$

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$\therefore S_{v\min} = \dfrac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$

$= \dfrac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$

$= 225.67mm.$

Maximum spacing of stirrups:

lxxxv) $0.75 \times d = 0.75 \times 450 = 337.5mm$

lxxxvi) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200mm$

lxxxvii) 225.67mm

∴ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxxxv) Depth of the member = 500mm

lxxxvi) $\frac{1}{6}$ (Clear span) = $\frac{1}{6} \times (4.5 - 0.45) = 0.675m = 675mm$

lxxxvii) 450mm.

$\therefore$ L_0 = 675 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4} \times 400 = 100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18 \times S \times h \times \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1 \right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 440 mm^2 (440 = 500 – 40 – 40 + 10 +10)

$$\therefore A_{sh} = 0.18 \times 100 \times 340 \times \frac{20}{415} \times \left(\frac{400 \times 500}{340 \times 440} - 1 \right)$$

= 99.36mm^2

Use 10Φ hook.

Spacing = $\frac{100 \times 78.53}{99.36} = 79.03mm$

Provide 10Φ hook @ 75mm c/c.

<u>Column No. 41, 71:</u>

Shear reinforcement:

Capacity based shear = 289.03KN.

Shear force from analysis = 148.58KN

$\therefore V_u = 289.03$KN

$\tau_c = 0.82 \times \delta$

Where $\delta = 1 + \dfrac{3P_u}{A_g f_{ck}}$

$= 1 + \dfrac{3 \times 1645.35 \times 10^3}{400 \times 550 \times 20}$

$= 2.122$

$\therefore \tau_c = 0.82 \times 2.122 = 1.74 \text{N/mm}^2$

$V_c = \tau_c \times bd$

$= 1.74 \times 400 \times 500$

$= 348 \times 10^3$N

$V_u < V_c$

Hence minimum shear reinforcement will be provided.

Use 8Φ two legged stirrup.

$\therefore S_{v\min} = \dfrac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$

$= \dfrac{0.87 \times 415 \times (2 \times 50)}{0.4 \times 400}$

$= 225.67$mm.

Maximum spacing of stirrups:

lxxxviii) $0.75 \times d = 0.75 \times 500 = 375$mm

lxxxix) $(1/2) \times$ Least lateral dimension $= (1/2) \times 400 = 200$mm

xc) 225.67mm

$\therefore$ Provide 8Φ two legged stirrup @ 200mm c/c.

Special confining reinforcement:

Special confining reinforcement will be provided over a length of L_0 from each joint face towards mid span.

L_0 shall not less than

lxxxviii) Depth of the member = 550mm

lxxxix) $\frac{1}{6}$ (Clear span) = $\frac{1}{6}\times(4.5-0.45)=0.675m=675mm$

xc) 450mm.

$\therefore$ L_0 = 675 mm

The spacing of hoop

i) Shall not exceed $\frac{1}{4}$ (Minimum member dimension) = $\frac{1}{4}\times 400=100mm$

ii) Should not less than 75mm

iii) Should not greater than 100mm.

$\therefore$ Spacing of hoops, S = 100mm

Minimum area of cross section of bar forming hoop, A_{sh}:

$$= 0.18\times S\times h\times\frac{f_{ck}}{f_y}\times\left(\frac{A_g}{A_k}-1\right)$$

Where

h = Larger dimension of the rectangular confining hoop measured to it's outer face and should not exceed 300mm.

= 400 – 40 – 40 + 10 +10 = 340mm.

(Assuming to use 10Φ hook)

A_k = Area of confined concrete in the rectangular hoop measured to it's out side dimension.

= 340 × 490 mm^2 (490 = 550 – 40 – 40 + 10 +10)

$$\therefore A_{sh}=0.18\times 100\times 340\times\frac{20}{415}\times\left(\frac{400\times 550}{340\times 490}-1\right)$$

= 94.53mm^2

Use 10Φ hook.

Spacing = $\frac{100\times 78.53}{94.53}=83.07mm$

Provide 10Φ hook @ 80mm c/c.

Appendix -2

Analysis Results

Analysis result of RC frame with EQ force in X direction:

Table of element forces:-

Frame	Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3
Text	Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
1	1	COMB1	Combination	5.887416403	4.470965719	791.8595834	-6.833031639	8.780756772	2.73E-02
1	2	COMB1	Combination	-5.887416403	-4.470965719	-766.5470834	-13.2863141	17.71261704	-2.73E-02
1	1	COMB2	Combination	-68.20701273	3.57442332	271.7640941	-5.462369096	-186.3954182	2.66E-02
1	2	COMB2	Combination	68.20701273	-3.57442332	-251.5140941	-10.62253584	-120.5361391	-2.66E-02
1	1	COMB3	Combination	77.62687897	3.579121832	995.2112394	-5.470481526	200.444629	1.72E-02
1	2	COMB3	Combination	-77.62687897	-3.579121832	-974.9612394	-10.63556672	148.8763263	-1.72E-02
1	1	COMB4	Combination	-86.1361373	3.704596834	227.273	-5.658331739	-234.3040203	1.95E-02
1	2	COMB4	Combination	86.1361373	-3.704596834	-201.9605	-11.01235402	-153.3085976	-1.95E-02
1	1	COMB5	Combination	96.15622733	3.710469974	1131.581932	-5.668472275	249.2460388	7.74E-03
1	2	COMB5	Combination	-96.15622733	-3.710469974	-1106.269432	-11.02864261	183.4569842	-7.74E-03
1	1	COMB6	Combination	-51.68168238	2.222758101	136.3638	-3.394999043	-140.5824122	1.17E-02
1	2	COMB6	Combination	51.68168238	-2.222758101	-121.1763	-6.60741241	-91.98515854	-1.17E-02
1	1	COMB7	Combination	57.6937364	2.226281985	678.949159	-3.401083365	149.5476233	4.65E-03
1	2	COMB7	Combination	-57.6937364	-2.226281985	-663.761659	-6.617185565	110.0741905	-4.65E-03
2	2	COMB1	Combination	18.27585335	14.033118	625.0503391	-25.44421509	33.63643003	-2.83E-02
2	3	COMB1	Combination	-18.27585335	-14.033118	-605.3628391	-23.67169792	30.32905669	2.83E-02
2	2	COMB2	Combination	-39.539086	11.21854905	245.2969121	-20.34203607	-60.10915017	-1.80E-02
2	3	COMB2	Combination	39.539086	-11.21854905	-229.5469121	-18.92288559	-78.27765082	1.80E-02
2	2	COMB3	Combination	68.78045136	11.23443976	754.7836305	-20.36870808	113.9274382	-2.73E-02
2	3	COMB3	Combination	-68.78045136	-11.23443976	-739.0336305	-18.95183109	126.8041415	2.73E-02
2	2	COMB4	Combination	-52.20935391	11.59372315	215.2781651	-21.04792924	-80.22629786	-8.86E-03
2	3	COMB4	Combination	52.20935391	-11.59372315	-195.5906651	-19.53010178	-102.5064408	8.86E-03
2	2	COMB5	Combination	83.19006778	11.61358654	852.1365631	-21.08126925	137.3194376	-2.05E-02
2	3	COMB5	Combination	-83.19006778	-11.61358654	-832.4490631	-19.56628365	153.8457996	2.05E-02
2	2	COMB6	Combination	-31.32561235	6.95623389	129.1668991	-12.62875755	-48.13577872	-5.31E-03
2	3	COMB6	Combination	31.32561235	-6.95623389	-117.3543991	-11.71806107	-61.5038645	5.31E-03
2	2	COMB7	Combination	49.91404067	6.968151927	511.2819379	-12.64876155	82.39166258	-1.23E-02
2	3	COMB7	Combination	-49.91404067	-6.968151927	-499.4694379	-11.73977019	92.30747977	1.23E-02
3	3	COMB1	Combination	16.81041199	13.61452457	457.356225	-23.51048424	28.78438433	-0.004405796
3	4	COMB1	Combination	-16.81041199	-13.61452457	-437.668725	-24.14035175	30.05205765	0.004405796
3	3	COMB2	Combination	-40.78890486	10.88113293	205.2716971	-18.79052812	-66.46948595	8.36E-04

3	4	COMB2	Combination	40.78890486	-10.88113293	-189.5216971	-19.29343714	-76.29168107	-8.36E-04
3	3	COMB3	Combination	67.68556405	10.90210638	526.4982628	-18.82624667	112.5245009	-7.89E-03
3	4	COMB3	Combination	-67.68556405	-10.90210638	-510.7482628	-19.33112567	124.3749733	7.89E-03
3	3	COMB4	Combination	-53.69992946	11.13297915	187.7384925	-19.25297989	-87.69588238	3.09E-03
3	4	COMB4	Combination	53.69992946	-11.13297915	-168.0509925	-19.71244714	-100.2538707	-3.09E-03
3	3	COMB5	Combination	81.89315669	11.15919596	589.2716996	-19.29762807	136.0466012	-7.81E-03
3	4	COMB5	Combination	-81.89315669	-11.15919596	-569.5841996	-19.7595578	150.5794472	7.81E-03
3	3	COMB6	Combination	-32.21995768	6.679787491	112.6430955	-11.55178793	-52.61752943	1.86E-03
3	4	COMB6	Combination	32.21995768	-6.679787491	-100.8305955	-11.82746828	-60.15232244	-1.86E-03
3	3	COMB7	Combination	49.13589401	6.695517579	353.5630198	-11.57857684	81.6279607	-4.68E-03
3	4	COMB7	Combination	-49.13589401	-6.695517579	-341.7505198	-11.85573468	90.34766834	4.68E-03
4	4	COMB1	Combination	17.23658507	14.36801322	287.4342073	-25.08145657	30.60885349	-1.51E-02
4	5	COMB1	Combination	-17.23658507	-14.36801322	-267.7467073	-25.20658968	29.71919427	1.51E-02
4	4	COMB2	Combination	-29.08806377	11.48257272	149.372964	-20.04453063	-38.98915178	-8.40E-03
4	5	COMB2	Combination	29.08806377	-11.48257272	-133.622964	-20.1444739	-62.81907142	8.40E-03
4	4	COMB3	Combination	56.66659989	11.50624842	310.5217677	-20.08579988	87.96331736	-1.57E-02
4	5	COMB3	Combination	-56.66659989	-11.50624842	-294.7717677	-20.1860696	110.3697822	1.57E-02
4	4	COMB4	Combination	-39.04559281	11.8027987	140.9888533	-20.51947677	-53.62086999	-9.28E-03
4	5	COMB4	Combination	39.04559281	-11.8027987	-121.3013533	-20.7903187	-83.03870483	9.28E-03
4	4	COMB5	Combination	68.14773677	11.83239333	342.424858	-20.57106332	105.0697164	-1.85E-02
4	5	COMB5	Combination	-68.14773677	-11.83239333	-322.737358	-20.84231333	133.4473623	1.85E-02
4	4	COMB6	Combination	-23.42735568	7.081679223	84.59331196	-12.31168606	-32.172522	-5.57E-03
4	5	COMB6	Combination	23.42735568	-7.081679223	-72.78081196	-12.47419122	-49.8232229	5.57E-03
4	4	COMB7	Combination	40.88864206	7.099435997	205.4549148	-12.34263799	63.04182986	-1.11E-02
4	5	COMB7	Combination	-40.88864206	-7.099435997	-193.6424148	-12.505388	80.06841735	1.11E-02
5	5	COMB1	Combination	20.46003242	15.16879072	115.9247639	-25.5906865	33.25953371	-5.09E-02
5	6	COMB1	Combination	-20.46003242	-15.16879072	-96.23726391	-27.50008101	38.35057977	5.09E-02
5	5	COMB2	Combination	-5.728083954	12.1202805	67.08764267	-20.44926003	1.264609205	-3.85E-02
5	6	COMB2	Combination	5.728083954	-12.1202805	-51.33764267	-21.97172172	-21.31290304	3.85E-02
5	5	COMB3	Combination	38.46413583	12.14978465	118.3919796	-20.49583837	51.95064473	-4.29E-02
5	6	COMB3	Combination	-38.46413583	-12.14978465	-102.6419796	-22.0284079	82.67383068	4.29E-02
5	5	COMB4	Combination	-11.30191076	11.65975366	61.98648589	-20.16462113	-4.629227217	-8.24E-03
5	6	COMB4	Combination	11.30191076	-11.65975366	-42.29898589	-20.64451668	-34.92746044	8.24E-03
5	5	COMB5	Combination	43.93836398	11.69663385	126.1169071	-20.22284405	58.72831719	-1.38E-02
5	6	COMB5	Combination	-43.93836398	-11.69663385	-106.4294071	-20.71537441	95.05595672	1.38E-02

5	5	COMB6	Combination	-6.781146455	6.995852196	37.19189154	-12.09877268	-2.77753633	-4.94E-03
5	6	COMB6	Combination	6.781146455	-6.995852196	-25.37939154	-12.38671001	-20.95647626	4.94E-03
5	5	COMB7	Combination	26.36301839	7.017980308	75.67014423	-12.13370643	35.23699032	-8.28E-03
5	6	COMB7	Combination	-26.36301839	-7.017980308	-63.85764423	-12.42922465	57.03357403	8.28E-03
6	7	COMB1	Combination	7.573603204	7.75E-02	1199.786298	-0.234429899	11.2963349	1.52E-02
6	8	COMB1	Combination	-7.573603204	-7.75E-02	-1174.473798	-0.114238335	22.78487952	-1.52E-02
6	7	COMB2	Combination	-67.14233029	5.98E-02	596.6566601	-0.183790348	-185.1406539	1.75E-02
6	8	COMB2	Combination	67.14233029	-5.98E-02	-576.4066601	-8.51E-02	-116.9998324	-1.75E-02
6	7	COMB3	Combination	79.26009541	0.064219272	1323.001417	-0.191297491	203.2147898	6.79E-03
6	8	COMB3	Combination	-79.26009541	-0.064219272	-1302.751417	-9.77E-02	153.4556396	-6.79E-03
6	7	COMB4	Combination	-85.67991771	1.93E-02	529.7110749	-0.124674377	-234.0410039	1.42E-02
6	8	COMB4	Combination	85.67991771	-1.93E-02	-504.3985749	3.78E-02	-151.5186258	-1.42E-02
6	7	COMB5	Combination	97.32311441	2.49E-02	1437.64202	-0.134058306	251.4033007	7.88E-04
6	8	COMB5	Combination	-97.32311441	-2.49E-02	-1412.32952	2.21E-02	186.5507142	-7.88E-04
6	7	COMB6	Combination	-51.40795063	1.16E-02	317.826645	-7.48E-02	-140.4246023	8.54E-03
6	8	COMB6	Combination	51.40795063	-1.16E-02	-302.639145	2.27E-02	-90.91117548	-8.54E-03
6	7	COMB7	Combination	58.39386865	1.49E-02	862.5852123	-8.04E-02	150.8419804	4.73E-04
6	8	COMB7	Combination	-58.39386865	-1.49E-02	-847.3977123	1.32E-02	111.9304285	-4.73E-04
7	8	COMB1	Combination	23.59752847	8.34E-02	937.6525433	0.286378093	43.38017319	-1.45E-02
7	9	COMB1	Combination	-23.59752847	-8.34E-02	-917.9650433	-0.578106398	39.21117644	1.45E-02
7	8	COMB2	Combination	-35.50348999	5.84E-02	494.3394948	0.242907795	-52.68072783	-6.34E-03
7	9	COMB2	Combination	35.50348999	-5.84E-02	-478.5894948	-0.447430645	-71.58148713	6.34E-03
7	8	COMB3	Combination	73.25953554	7.49E-02	1005.904575	0.215297155	122.0890049	-1.69E-02
7	9	COMB3	Combination	-73.25953554	-7.49E-02	-990.1545746	-0.477539592	134.3193694	1.69E-02
7	8	COMB4	Combination	-49.93609226	-0.128507226	444.5129104	0.566506279	-76.00834753	-8.22E-04
7	9	COMB4	Combination	49.93609226	0.128507226	-424.8254104	-0.116730989	-98.76797539	8.22E-04
7	8	COMB5	Combination	86.01768964	-0.107893088	1083.96926	0.531992979	142.4538184	-1.40E-02
7	9	COMB5	Combination	-86.01768964	0.107893088	-1064.28176	-0.154367173	158.6080953	1.40E-02
7	8	COMB6	Combination	-29.96165536	-7.71E-02	266.7077462	0.339903767	-45.60500852	-4.93E-04
7	9	COMB6	Combination	29.96165536	7.71E-02	-254.8952462	-7.00E-02	-59.26078523	4.93E-04
7	8	COMB7	Combination	51.61061379	-6.47E-02	650.3815561	0.319195787	85.47229106	-8.38E-03
7	9	COMB7	Combination	-51.61061379	6.47E-02	-638.5690561	-9.26E-02	95.16485719	8.38E-03
8	9	COMB1	Combination	21.74364079	1.552966631	681.1520943	-2.684845188	37.2657412	-3.32E-03
8	10	COMB1	Combination	-21.74364079	-1.552966631	-661.4645943	-2.750538021	38.83700158	3.32E-03
8	9	COMB2	Combination	-37.07878963	1.231519945	383.6362575	-2.129371298	-60.08655256	2.21E-03

8	10	COMB2	Combination	37.07878963	-1.231519945	-367.8862575	-2.180948508	-69.68921113	-2.21E-03
8	9	COMB3	Combination	71.8686149	1.253226665	706.2070933	-2.166381003	119.7117385	-7.52E-03
8	10	COMB3	Combination	-71.8686149	-1.253226665	-690.4570933	-2.219912326	131.8284137	7.52E-03
8	9	COMB4	Combination	-51.75114751	1.040592174	349.2374204	-1.799841933	-84.28497368	4.10E-03
8	10	COMB4	Combination	51.75114751	-1.040592174	-329.5499204	-1.842230676	-96.84404261	-4.10E-03
8	9	COMB5	Combination	84.43310814	1.067725575	752.4509652	-1.846104063	140.4628901	-8.06E-03
8	10	COMB5	Combination	-84.43310814	-1.067725575	-732.7634652	-1.890935449	155.0529884	8.06E-03
8	9	COMB6	Combination	-31.05068851	0.624355304	209.5424523	-1.07990516	-50.57098421	2.46E-03
8	10	COMB6	Combination	31.05068851	-0.624355304	-197.7299523	-1.105338406	-58.10642557	-2.46E-03
8	9	COMB7	Combination	50.65986488	0.640635345	451.4705791	-1.107662438	84.27773407	-4.83E-03
8	10	COMB7	Combination	-50.65986488	-0.640635345	-439.6580791	-1.134561269	93.03179302	4.83E-03
9	10	COMB1	Combination	22.63733905	1.938539859	424.3275215	-3.233157058	39.98144591	-8.65E-03
9	11	COMB1	Combination	-22.63733905	-1.938539859	-404.6400215	-3.551732449	39.24924077	8.65E-03
9	10	COMB2	Combination	-24.97177014	1.538326073	258.5412828	-2.564888233	-31.80931882	-2.88E-03
9	11	COMB2	Combination	24.97177014	-1.538326073	-242.7912828	-2.819253024	-55.59187666	2.88E-03
9	10	COMB3	Combination	61.19151262	1.563337701	420.3827516	-2.608163061	95.77963227	-1.10E-02
9	11	COMB3	Combination	-61.19151262	-1.563337701	-404.6327516	-2.863518894	118.3906619	1.10E-02
9	10	COMB4	Combination	-36.55487222	1.219176841	236.2526908	-2.044217429	-49.47660912	-2.12E-03
9	11	COMB4	Combination	36.55487222	-1.219176841	-216.5651908	-2.222901513	-78.46544364	2.12E-03
9	10	COMB5	Combination	71.14923123	1.250441375	438.5545269	-2.098310964	110.0095797	-1.22E-02
9	11	COMB5	Combination	-71.14923123	-1.250441375	-418.8670269	-2.27823385	139.0127296	1.22E-02
9	10	COMB6	Combination	-21.93292333	0.731506104	141.7516145	-1.226530458	-29.68596547	-1.27E-03
9	11	COMB6	Combination	21.93292333	-0.731506104	-129.9391145	-1.333740908	-47.07926618	1.27E-03
9	10	COMB7	Combination	42.68953874	0.750264825	263.1327161	-1.258986579	66.00574784	-7.33E-03
9	11	COMB7	Combination	-42.68953874	-0.750264825	-251.3202161	-1.36694031	83.40763774	7.33E-03
10	11	COMB1	Combination	25.91673698	1.80164469	167.1405599	-2.975193307	42.7076737	-3.00E-02
10	12	COMB1	Combination	-25.91673698	-1.80164469	-147.4530599	-3.330563109	48.00090574	3.00E-02
10	11	COMB2	Combination	-1.494064076	1.427224856	107.9494594	-2.356914007	8.653723905	-0.021492778
10	12	COMB2	Combination	1.494064076	-1.427224856	-92.19945941	-2.63837299	-13.88294817	0.021492778
10	11	COMB3	Combination	42.96084325	1.455406648	159.4754364	-2.403395284	59.67855402	-2.64E-02
10	12	COMB3	Combination	-42.96084325	-1.455406648	-143.7254364	-2.690527984	90.68439736	2.64E-02
10	11	COMB4	Combination	-10.10895961	1.240595903	91.69023608	-1.978610533	-1.536508425	-4.59E-03
10	12	COMB4	Combination	10.10895961	-1.240595903	-72.00273608	-2.363475129	-33.84485023	4.59E-03
10	11	COMB5	Combination	45.45967454	1.275823143	156.0977073	-2.036712128	62.24452922	-0.010766707
10	12	COMB5	Combination	-45.45967454	-1.275823143	-136.4102073	-2.428668871	96.86433168	0.010766707

10	11	COMB6	Combination	-6.065375769	0.744357542	55.01414165	-1.18716632	-0.921905055	-2.76E-03
10	12	COMB6	Combination	6.065375769	-0.744357542	-43.20164165	-1.418085077	-20.30691014	2.76E-03
10	11	COMB7	Combination	27.27580473	0.765493886	93.65862438	-1.222027277	37.34671753	-6.46E-03
10	12	COMB7	Combination	-27.27580473	-0.765493886	-81.84612438	-1.457201323	58.11859901	6.46E-03
31	37	COMB1	Combination	-2.348942997	5.832708022	1105.65514	-8.931252809	-3.397743436	1.82E-02
31	38	COMB1	Combination	2.348942997	-5.832708022	-1077.81139	-17.31593329	-7.172500051	-1.82E-02
31	37	COMB2	Combination	-118.3666426	4.665274733	684.6043026	-7.143706793	-288.1653718	1.54E-02
31	38	COMB2	Combination	118.3666426	-4.665274733	-662.3293026	-13.8500295	-244.4845197	-1.54E-02
31	37	COMB3	Combination	114.6083338	4.667058102	1084.443921	-7.146297702	282.7289823	1.37E-02
31	38	COMB3	Combination	-114.6083338	-4.667058102	-1062.168921	-13.85546376	233.0085196	-1.37E-02
31	37	COMB4	Combination	-147.6265241	4.435991458	673.4784675	-6.789455095	-359.7278944	1.01E-02
31	38	COMB4	Combination	147.6265241	-4.435991458	-645.6347175	-13.17250646	-304.5914641	-1.01E-02
31	37	COMB5	Combination	143.5921963	4.43822067	1173.277991	-6.792693731	353.8900483	8.08E-03
31	38	COMB5	Combination	-143.5921963	-4.43822067	-1145.434241	-13.17929928	292.274835	-8.08E-03
31	37	COMB6	Combination	-88.57591447	2.661594875	404.0870805	-4.073673057	-215.8367366	6.07E-03
31	38	COMB6	Combination	88.57591447	-2.661594875	-387.3808305	-7.903503879	-182.7548785	-6.07E-03
31	37	COMB7	Combination	86.15531777	2.662932402	703.9667946	-4.075616239	212.334029	4.85E-03
31	38	COMB7	Combination	-86.15531777	-2.662932402	-687.2605446	-7.90757957	175.364901	-4.85E-03
32	38	COMB1	Combination	-6.723213287	18.0762371	861.6644385	-32.81594242	-13.11986587	-2.08E-02
32	39	COMB1	Combination	6.723213287	-18.0762371	-840.0081885	-30.45088742	-10.41138064	2.08E-02
32	38	COMB2	Combination	-135.0539469	14.45724348	554.2911625	-26.24647522	-235.6979997	-1.61E-02
32	39	COMB2	Combination	135.0539469	-14.45724348	-536.9661625	-24.35387695	-236.9908146	1.61E-02
32	38	COMB3	Combination	124.2968057	14.46473588	824.371939	-26.25903266	214.7062143	-1.73E-02
32	39	COMB3	Combination	-124.2968057	-14.46473588	-807.046939	-24.36754292	220.3326056	1.73E-02
32	38	COMB4	Combination	-167.9463617	13.69036109	546.6315929	-24.89132572	-292.8655965	-9.92E-03
32	39	COMB4	Combination	167.9463617	-13.69036109	-524.9753429	-23.02493809	-294.9466695	9.92E-03
32	38	COMB5	Combination	156.242079	13.69972659	884.2325636	-24.90702252	270.139671	-1.15E-02
32	39	COMB5	Combination	-156.242079	-13.69972659	-862.5763136	-23.04202056	276.7076057	1.15E-02
32	38	COMB6	Combination	-100.767817	8.214216653	327.9789558	-14.93479543	-175.7193579	-5.95E-03
32	39	COMB6	Combination	100.767817	-8.214216653	-314.9852058	-13.81496285	-176.9680017	5.95E-03
32	38	COMB7	Combination	93.74524743	8.219835957	530.5395381	-14.94421351	162.0838026	-6.87E-03
32	39	COMB7	Combination	-93.74524743	-8.219835957	-517.5457881	-13.82521233	166.0245634	6.87E-03
33	39	COMB1	Combination	-4.318517954	17.49133498	622.6826359	-30.22323555	-7.305287439	-1.14E-04
33	40	COMB1	Combination	4.318517954	-17.49133498	-601.0263859	-30.9964369	-7.809525401	1.14E-04
33	39	COMB2	Combination	-115.421902	13.98808787	424.5808327	-24.17007721	-195.0218117	5.45E-04

33	40	COMB2	Combination	115.421902	-13.98808787	-407.2558327	-24.78823033	-208.9548454	-5.45E-04
33	39	COMB3	Combination	108.5122733	13.99804811	571.7113847	-24.18709966	183.3333518	-7.28E-04
33	40	COMB3	Combination	-108.5122733	-13.99804811	-554.3863847	-24.80606871	196.4596047	7.28E-04
33	39	COMB4	Combination	-143.8371526	13.10091937	421.0807508	-22.67269063	-243.0383437	9.23E-04
33	40	COMB4	Combination	143.8371526	-13.10091937	-399.4245008	-23.18052716	-260.3916906	-9.23E-04
33	39	COMB5	Combination	136.0805665	13.11336966	604.9939409	-22.69396869	229.9056107	-6.68E-04
33	40	COMB5	Combination	-136.0805665	-13.11336966	-583.3376909	-23.20282513	246.3763721	6.68E-04
33	39	COMB6	Combination	-86.30229158	7.860551621	252.6484505	-13.60361438	-145.8230062	5.54E-04
33	40	COMB6	Combination	86.30229158	-7.860551621	-239.6547005	-13.9083163	-156.2350143	-5.54E-04
33	39	COMB7	Combination	81.6483399	7.868021797	362.9963645	-13.61638121	137.9433664	-4.01E-04
33	40	COMB7	Combination	-81.6483399	-7.868021797	-350.0026145	-13.92169508	147.8258232	4.01E-04
34	40	COMB1	Combination	-3.627135815	18.48970991	383.2606971	-32.22169019	-6.93628418	-6.71E-03
34	41	COMB1	Combination	3.627135815	-18.48970991	-361.6044471	-32.49229451	-5.758691175	6.71E-03
34	40	COMB2	Combination	-90.01586788	14.78618406	278.0247881	-25.76755644	-146.6666001	-4.78E-03
34	41	COMB2	Combination	90.01586788	-14.78618406	-260.6997881	-25.98408776	-168.3889375	4.78E-03
34	40	COMB3	Combination	84.21245058	14.79735181	335.1923272	-25.78714787	135.5685454	-5.95E-03
34	41	COMB3	Combination	-84.21245058	-14.79735181	-317.8673272	-26.00358346	159.1750317	5.95E-03
34	40	COMB4	Combination	-112.3405382	13.9852997	274.6688656	-24.21409624	-182.8697816	-7.83E-03
34	41	COMB4	Combination	112.3405382	-13.9852997	-253.0126156	-24.73445272	-210.3221022	7.83E-03
34	40	COMB5	Combination	105.4448599	13.99925939	346.1282894	-24.23858553	169.9241502	-9.29E-03
34	41	COMB5	Combination	-105.4448599	-13.99925939	-324.4720394	-24.75882235	199.1328593	9.29E-03
34	40	COMB6	Combination	-67.40432293	8.391179821	164.8013193	-14.52845774	-109.721869	-4.70E-03
34	41	COMB6	Combination	67.40432293	-8.391179821	-151.8075693	-14.84067163	-126.1932613	4.70E-03
34	40	COMB7	Combination	63.26691592	8.399555636	207.6769736	-14.54315132	101.9544901	-0.005574667
34	41	COMB7	Combination	-63.26691592	-8.399555636	-194.6832236	-14.85529341	119.4797156	0.005574667
35	41	COMB1	Combination	-5.752810451	19.03348047	143.9244557	-32.33962325	-8.585589785	-3.42E-02
35	42	COMB1	Combination	5.752810451	-19.03348047	-122.2682057	-34.2775584	-11.54924679	3.42E-02
35	41	COMB2	Combination	-49.79061745	15.21980295	112.4139342	-25.86061781	-71.92152296	-2.74E-02
35	42	COMB2	Combination	49.79061745	-15.21980295	-95.0889342	-27.40869252	-102.3456381	2.74E-02
35	41	COMB3	Combination	40.58612072	15.2337658	117.8651949	-25.88277939	58.1845793	-2.73E-02
35	42	COMB3	Combination	-40.58612072	-15.2337658	-100.5401949	-27.43540091	83.86684323	2.73E-02
35	41	COMB4	Combination	-61.65290944	12.95608832	104.5784177	-22.92569452	-89.17687991	-6.66E-03
35	42	COMB4	Combination	61.65290944	-12.95608832	-82.92216769	-22.42061461	-126.6083031	6.66E-03
35	41	COMB5	Combination	51.31801327	12.97354188	111.3924935	-22.9533965	73.45574791	-6.62E-03
35	42	COMB5	Combination	-51.31801327	-12.97354188	-89.73624354	-22.45400009	106.1572985	6.62E-03

35	41	COMB6	Combination	-36.99174567	7.773652993	62.74705061	-13.75541671	-53.50612794	-4.00E-03
35	42	COMB6	Combination	36.99174567	-7.773652993	-49.75330061	-13.45236876	-75.96498189	4.00E-03
35	41	COMB7	Combination	30.79080796	7.784125129	66.83549612	-13.7720379	44.07344875	-3.97E-03
35	42	COMB7	Combination	-30.79080796	-7.784125129	-53.84174612	-13.47240005	63.69437911	3.97E-03
36	43	COMB1	Combination	-3.064598713	0.133296933	1645.348181	-0.363225955	-4.433580561	1.07E-02
36	44	COMB1	Combination	3.064598713	-0.133296933	-1617.504431	-0.236610244	-9.357113646	-1.07E-02
36	43	COMB2	Combination	-119.3947452	0.10576866	1115.536256	-0.28931396	-290.1142522	9.55E-03
36	44	COMB2	Combination	119.3947452	-0.10576866	-1093.261256	-0.186645009	-247.1621013	-9.55E-03
36	43	COMB3	Combination	114.4913873	0.107506433	1517.020833	-0.291847568	283.0205233	7.51E-03
36	44	COMB3	Combination	-114.4913873	-0.107506433	-1494.745833	-0.191931382	232.1907195	-7.51E-03
36	43	COMB4	Combination	-148.5774856	5.09E-02	1040.661009	-0.198608501	-361.6815292	6.56E-03
36	44	COMB4	Combination	148.5774856	-5.09E-02	-1012.817259	-3.03E-02	-306.9171559	-6.56E-03
36	43	COMB5	Combination	143.7801801	5.30E-02	1542.51673	-0.201775512	354.7369401	4.00E-03
36	44	COMB5	Combination	-143.7801801	-5.30E-02	-1514.67298	-3.69E-02	292.2738701	-4.00E-03
36	43	COMB6	Combination	-89.14649134	3.05E-02	624.3966055	-0.119165101	-217.0089175	3.93E-03
36	44	COMB6	Combination	89.14649134	-3.05E-02	-607.6903555	-1.82E-02	-184.1502935	-3.93E-03
36	43	COMB7	Combination	86.26810804	0.031827267	925.5100382	-0.121065307	212.8421641	2.40E-03
36	44	COMB7	Combination	-86.26810804	-0.031827267	-908.8037882	-2.22E-02	175.3643221	-2.40E-03
37	44	COMB1	Combination	-8.757150446	9.77E-02	1274.060627	0.371497302	-17.09363171	-1.12E-02
37	45	COMB1	Combination	8.757150446	-9.77E-02	-1252.404377	-0.71342365	-13.55639485	1.12E-02
37	44	COMB2	Combination	-137.2110976	0.074305239	883.6361118	0.303619966	-239.8073344	-8.10E-03
37	45	COMB2	Combination	137.2110976	-0.074305239	-866.3111118	-0.563688301	-240.431507	8.10E-03
37	44	COMB3	Combination	123.1996568	8.20E-02	1154.860892	0.290775718	212.4575237	-9.82E-03
37	45	COMB3	Combination	-123.1996568	-8.20E-02	-1137.535892	-0.577789539	218.7412752	9.82E-03
37	44	COMB4	Combination	-169.758211	-0.146107894	824.2012846	0.652117307	-296.2242644	-4.59E-03
37	45	COMB4	Combination	169.758211	0.146107894	-802.5450346	-0.140739677	-297.9294742	4.59E-03
37	44	COMB5	Combination	155.755232	-0.136484506	1163.23226	0.636061997	269.1068082	-6.74E-03
37	45	COMB5	Combination	-155.755232	0.136484506	-1141.57601	-0.158366225	276.0365036	6.74E-03
37	44	COMB6	Combination	-101.8549266	-8.77E-02	494.5207707	0.391270384	-177.7345587	-2.75E-03
37	45	COMB6	Combination	101.8549266	8.77E-02	-481.5270207	-8.44E-02	-178.7576845	2.75E-03
37	44	COMB7	Combination	93.45313917	-8.19E-02	697.9393558	0.381637198	161.4640849	-4.04E-03
37	45	COMB7	Combination	-93.45313917	8.19E-02	-684.9456058	-9.50E-02	165.6219022	4.04E-03
38	45	COMB1	Combination	-5.421587224	1.986113461	918.0090777	-3.43937923	-9.26012832	-9.71E-04
38	46	COMB1	Combination	5.421587224	-1.986113461	-896.3528277	-3.512017884	-9.715426964	9.71E-04
38	45	COMB2	Combination	-116.7951433	1.583759384	660.5145661	-2.742730261	-197.4283559	6.60E-05

38	46	COMB2	Combination	116.7951433	-1.583759384	-643.1895661	-2.800427582	-211.3546457	-6.60E-05
38	45	COMB3	Combination	108.1206038	1.594022154	808.2999583	-2.760276507	182.6121506	-1.62E-03
38	46	COMB3	Combination	-108.1206038	-1.594022154	-790.9749583	-2.818801032	195.8099625	1.62E-03
38	45	COMB4	Combination	-145.090064	1.254602914	616.055889	-2.16965814	-245.2688474	4.67E-04
38	46	COMB4	Combination	145.090064	-1.254602914	-594.399639	-2.221452057	-262.5463765	-4.67E-04
38	45	COMB5	Combination	136.0546199	1.267431376	800.7876293	-2.191590948	229.7817858	-1.64E-03
38	46	COMB5	Combination	-136.0546199	-1.267431376	-779.1313793	-2.244418869	246.4093838	1.64E-03
38	45	COMB6	Combination	-87.05403837	0.752761748	369.6335334	-1.301794884	-147.1613084	2.80E-04
38	46	COMB6	Combination	87.05403837	-0.752761748	-356.6397834	-1.332871234	-157.5278259	-2.80E-04
38	45	COMB7	Combination	81.63277194	0.760458826	480.4725776	-1.314954569	137.8690715	-9.84E-04
38	46	COMB7	Combination	-81.63277194	-0.760458826	-467.4788276	-1.346651322	147.8456303	9.84E-04
39	46	COMB1	Combination	-5.018556092	2.416978266	564.8870523	-4.037687163	-9.187720541	-0.004340573
39	47	COMB1	Combination	5.018556092	-2.416978266	-543.2308023	-4.42173677	-8.37722578	0.004340573
39	46	COMB2	Combination	-91.54927093	1.92771224	423.1901825	-3.219939742	-149.1646615	-2.74E-03
39	47	COMB2	Combination	91.54927093	-1.92771224	-405.8651825	-3.527053097	-171.2577868	2.74E-03
39	46	COMB3	Combination	83.51958118	1.939452987	480.6291012	-3.240359718	134.4643086	-4.21E-03
39	47	COMB3	Combination	-83.51958118	-1.939452987	-463.3041012	-3.547725736	157.8542255	4.21E-03
39	46	COMB4	Combination	-114.0485821	1.364681109	389.0743416	-2.310837219	-185.4779286	-3.74E-03
39	47	COMB4	Combination	114.0485821	-1.364681109	-367.4180916	-2.465546664	-213.6921088	3.74E-03
39	46	COMB5	Combination	104.787483	1.379357043	460.8729901	-2.336362188	169.058284	-5.58E-03
39	47	COMB5	Combination	-104.787483	-1.379357043	-439.2167401	-2.491387463	197.6979066	5.58E-03
39	46	COMB6	Combination	-68.42914927	0.818808666	233.444605	-1.386502331	-111.2867572	-2.24E-03
39	47	COMB6	Combination	68.42914927	-0.818808666	-220.450855	-1.479327998	-128.2152653	2.24E-03
39	46	COMB7	Combination	62.87248982	0.827614226	276.523794	-1.401817313	101.4349704	-3.35E-03
39	47	COMB7	Combination	-62.87248982	-0.827614226	-263.530044	-1.494832478	118.618744	3.35E-03
40	47	COMB1	Combination	-5.147971181	2.339799036	214.2585672	-3.795230904	-8.76408661	-2.12E-02
40	48	COMB1	Combination	5.147971181	-2.339799036	-192.6023172	-4.394065721	-9.253812524	2.12E-02
40	47	COMB2	Combination	-49.57423357	1.865147763	168.6679653	-3.025133025	-72.46570019	-1.68E-02
40	48	COMB2	Combination	49.57423357	-1.865147763	-151.3429653	-3.502884146	-101.0441173	1.68E-02
40	47	COMB3	Combination	41.33747968	1.878530694	174.1457422	-3.047236421	58.44316162	-1.72E-02
40	48	COMB3	Combination	-41.33747968	-1.878530694	-156.8207422	-3.527621009	86.23801726	1.72E-02
40	47	COMB4	Combination	-60.75873367	1.602382616	139.1375858	-2.470620285	-89.07000762	-4.78E-03
40	48	COMB4	Combination	60.75873367	-1.602382616	-117.4813358	-3.137718872	-123.5855602	4.78E-03
40	47	COMB5	Combination	52.88090789	1.61911128	145.9848069	-2.49824953	74.56606963	-5.27E-03
40	48	COMB5	Combination	-52.88090789	-1.61911128	-124.3285569	-3.16863995	110.517108	5.27E-03

40	47	COMB6	Combination	-36.4552402	0.96142957	83.48255146	-1.482372171	-53.44200457	-2.87E-03
40	48	COMB6	Combination	36.4552402	-0.96142957	-70.48880146	-1.882631323	-74.15133613	2.87E-03
40	47	COMB7	Combination	31.72854474	0.971466768	87.59088416	-1.498949718	44.73964178	-3.16E-03
40	48	COMB7	Combination	-31.72854474	-0.971466768	-74.59713416	-1.90118397	66.3102648	3.16E-03
121	2	COMB1	Combination	-12.39398794	4.04E-02	76.03047325	0.46201171	-51.34880412	6.74E-02
121	38	COMB1	Combination	12.39398794	-4.04E-02	77.71952675	-0.46201171	54.72691111	9.44E-02
121	2	COMB2	Combination	-18.62265257	2.22E-02	-46.14576516	0.369995417	180.6455171	0.029212861
121	38	COMB2	Combination	18.62265257	-2.22E-02	169.1457652	-0.369995417	249.9375435	5.95E-02
121	2	COMB3	Combination	-1.207728134	4.25E-02	167.7945224	0.369223318	-262.8036037	7.86E-02
121	38	COMB3	Combination	1.207728134	-4.25E-02	-44.79452236	-0.369223318	-162.3744858	9.15E-02
121	2	COMB4	Combination	-21.36734944	7.63E-03	-68.45018216	0.176382259	233.5350544	3.14E-03
121	38	COMB4	Combination	21.36734944	-7.63E-03	201.2001822	-0.176382259	305.7656743	2.74E-02
121	2	COMB5	Combination	0.401306104	3.31E-02	198.9751772	0.175417135	-320.7763466	6.49E-02
121	38	COMB5	Combination	-0.401306104	-3.31E-02	-66.22517725	-0.175417135	-209.6243623	6.75E-02
121	2	COMB6	Combination	-12.82040966	4.58E-03	-41.07010929	0.105829355	140.1210326	1.88E-03
121	38	COMB6	Combination	12.82040966	-4.58E-03	120.7201093	-0.105829355	183.4594046	1.64E-02
121	2	COMB7	Combination	0.240783663	1.99E-02	119.3851063	0.105250281	-192.465808	3.89E-02
121	38	COMB7	Combination	-0.240783663	-1.99E-02	-39.73510635	-0.105250281	-125.7746174	4.05E-02
122	3	COMB1	Combination	1.464653578	-9.20E-03	79.07719015	0.335296132	-59.11326667	-2.23E-02
122	39	COMB1	Combination	-1.464653578	9.20E-03	74.67280985	-0.335296132	50.30450606	-0.014507487
122	3	COMB2	Combination	32.72695716	-2.54E-02	-30.85194327	0.26883537	144.7473063	-6.18E-02
122	39	COMB2	Combination	-32.72695716	2.54E-02	153.8519433	-0.26883537	224.6604668	-0.039936079
122	3	COMB3	Combination	-30.38351144	1.07E-02	157.3754475	0.267638441	-239.328533	2.61E-02
122	39	COMB3	Combination	30.38351144	-1.07E-02	-34.37544752	-0.267638441	-144.1732571	1.67E-02
122	3	COMB4	Combination	40.8374995	-2.76E-02	-49.98228923	0.116339883	190.202444	-6.66E-02
122	39	COMB4	Combination	-40.8374995	2.76E-02	182.7322892	-0.116339883	275.226713	-4.38E-02
122	3	COMB5	Combination	-38.05058625	1.76E-02	185.3019493	0.114843721	-289.8923552	0.04324268
122	39	COMB5	Combination	38.05058625	-1.76E-02	-52.55194925	-0.114843721	-185.8154418	0.027046839
122	3	COMB6	Combination	24.5024997	-1.66E-02	-29.98937354	6.98E-02	114.1214664	-4.00E-02
122	39	COMB6	Combination	-24.5024997	1.66E-02	109.6393735	-6.98E-02	165.1360278	-2.63E-02
122	3	COMB7	Combination	-22.83035175	1.05E-02	111.1811696	6.89E-02	-173.9354131	2.59E-02
122	39	COMB7	Combination	22.83035175	-1.05E-02	-31.53116955	-6.89E-02	-111.4892651	1.62E-02
123	4	COMB1	Combination	-0.426661903	6.01E-03	80.12081298	0.359812308	-60.66071261	1.17E-02
123	40	COMB1	Combination	0.426661903	-6.01E-03	73.62918702	-0.359812308	47.6774607	1.24E-02
123	4	COMB2	Combination	53.32781237	-2.07E-02	-15.92217064	0.288654433	115.2810199	-5.27E-02

123	40	COMB2	Combination	-53.32781237	2.07E-02	138.9221706	-0.288654433	194.4076627	-3.01E-02
123	4	COMB3	Combination	-54.01047141	3.03E-02	144.1154714	0.28704526	-212.3381601	7.14E-02
123	40	COMB3	Combination	54.01047141	-3.03E-02	-21.1154714	-0.28704526	-118.1237256	4.98E-02
123	4	COMB4	Combination	66.63173827	-0.026040188	-31.62702243	0.117079073	153.8748614	-6.55E-02
123	40	COMB4	Combination	-66.63173827	0.026040188	164.3770224	-0.117079073	238.1332283	-3.86E-02
123	4	COMB5	Combination	-67.54111646	3.77E-02	168.4200301	0.115067607	-255.6491135	8.95E-02
123	40	COMB5	Combination	67.54111646	-3.77E-02	-35.67003012	-0.115067607	-152.531007	6.13E-02
123	4	COMB6	Combination	39.97904296	-1.56E-02	-18.97621346	7.02E-02	92.32491684	-3.93E-02
123	40	COMB6	Combination	-39.97904296	1.56E-02	98.62621346	-7.02E-02	142.879937	-0.023178135
123	4	COMB7	Combination	-40.52466988	2.26E-02	101.0520181	0.069040564	-153.3894681	5.37E-02
123	40	COMB7	Combination	40.52466988	-2.26E-02	-21.40201807	-0.069040564	-91.51860417	3.68E-02
124	5	COMB1	Combination	-3.222330089	1.28E-02	81.00138455	0.400381532	-62.97851692	3.33E-02
124	41	COMB1	Combination	3.222330089	-1.28E-02	72.74861545	-0.400381532	46.47297872	1.80E-02
124	5	COMB2	Combination	87.25363368	-2.15E-02	9.90116885	0.321179204	61.55465302	-0.050538484
124	41	COMB2	Combination	-87.25363368	2.15E-02	113.0988312	-0.321179204	144.8406716	-3.54E-02
124	5	COMB3	Combination	-92.40936182	4.20E-02	119.7010464	0.319431247	-162.3202801	0.103857206
124	41	COMB3	Combination	92.40936182	-4.20E-02	3.29895357	-0.319431247	-70.48390563	6.42E-02
124	5	COMB4	Combination	110.5240366	-4.61E-02	0.269264518	0.184948885	87.66808687	-0.103258722
124	41	COMB4	Combination	-110.5240366	4.61E-02	132.4807355	-0.184948885	176.7548551	-8.12E-02
124	5	COMB5	Combination	-114.0547078	3.32E-02	137.5191115	0.182763939	-192.1755795	8.97E-02
124	41	COMB5	Combination	114.0547078	-3.32E-02	-4.769111493	-0.182763939	-92.40086646	4.32E-02
124	5	COMB6	Combination	66.31442196	-2.77E-02	0.161558711	0.110969331	52.60085212	-6.20E-02
124	41	COMB6	Combination	-66.31442196	2.77E-02	79.48844129	-0.110969331	106.052913	-4.87E-02
124	5	COMB7	Combination	-68.43282467	1.99E-02	82.5114669	0.109658363	-115.3053477	5.38E-02
124	41	COMB7	Combination	68.43282467	-1.99E-02	-2.861466896	-0.109658363	-55.44051987	2.59E-02
125	6	COMB1	Combination	20.47003363	-4.95E-02	53.42209304	0.350829046	-38.35035261	-0.071992796
125	42	COMB1	Combination	-20.47003363	4.95E-02	47.82790696	-0.350829046	27.16198045	-0.12597009
125	6	COMB2	Combination	114.1456022	-7.54E-02	17.10112443	0.282539841	21.3130971	-0.144833295
125	42	COMB2	Combination	-114.1456022	7.54E-02	63.89887557	-0.282539841	72.28240519	-0.156816573
125	6	COMB3	Combination	-81.3935484	-3.77E-03	68.37422444	0.278786632	-82.67366129	2.96E-02
125	42	COMB3	Combination	81.3935484	3.77E-03	12.62577556	-0.278786632	-28.82323647	-4.47E-02
125	6	COMB4	Combination	138.5338344	-5.95E-02	10.08959241	-0.130238045	34.92745524	-0.127753393
125	42	COMB4	Combination	-138.5338344	5.95E-02	70.16040759	0.130238045	85.21417513	-0.11039408
125	6	COMB5	Combination	-105.8901038	3.00E-02	74.18096743	-0.134929557	-95.05599275	9.03E-02
125	42	COMB5	Combination	105.8901038	-3.00E-02	6.069032575	0.134929557	-41.16787695	2.97E-02

125	6	COMB6	Combination	83.12030065	-3.57E-02	6.053755445	-7.81E-02	20.95647314	-7.67E-02
125	42	COMB6	Combination	-83.12030065	3.57E-02	42.09624455	7.81E-02	51.12850508	-6.62E-02
125	6	COMB7	Combination	-63.53406231	1.80E-02	44.50858046	-8.10E-02	-57.03359565	5.42E-02
125	42	COMB7	Combination	63.53406231	-1.80E-02	3.641419545	8.10E-02	-24.70072617	0.017824303
126	38	COMB1	Combination	-8.022336806	1.44E-15	53.71875	-7.32E-18	-34.434628	-4.98E-02
126	74	COMB1	Combination	8.022336806	-1.44E-15	53.71875	7.32E-18	34.434628	0.049773617
126	38	COMB2	Combination	-1.453728813	-6.76E-03	-128.8695477	1.39E-03	230.2449342	-5.00E-02
126	74	COMB2	Combination	1.453728813	6.76E-03	214.8195477	-1.39E-03	285.288709	2.97E-02
126	38	COMB3	Combination	-11.38201008	0.006764982	214.8195477	-1.39E-03	-285.340339	-2.97E-02
126	74	COMB3	Combination	11.38201008	-0.006764982	-128.8695477	1.39E-03	-230.1933042	5.00E-02
126	38	COMB4	Combination	-0.444148697	-8.46E-03	-166.9931846	1.74E-03	291.6913731	-3.76E-02
126	74	COMB4	Combination	0.444148697	8.46E-03	262.6181846	-1.74E-03	352.7256808	1.22E-02
126	38	COMB5	Combination	-12.85450028	8.46E-03	262.6181846	-1.74E-03	-352.7902183	-1.22E-02
126	74	COMB5	Combination	12.85450028	-8.46E-03	-166.9931846	1.74E-03	-291.6268356	3.76E-02
126	38	COMB6	Combination	-0.266489218	-5.07E-03	-100.1959108	1.04E-03	175.0148239	-2.25E-02
126	74	COMB6	Combination	0.266489218	5.07E-03	157.5709108	-1.04E-03	211.6354085	7.33E-03
126	38	COMB7	Combination	-7.712700166	5.07E-03	157.5709108	-1.04E-03	-211.674131	-7.33E-03
126	74	COMB7	Combination	7.712700166	-5.07E-03	-100.1959108	1.04E-03	-174.9761014	2.25E-02
127	39	COMB1	Combination	-0.940429572	8.22E-16	53.71875	7.83E-16	-32.58787122	-5.43E-03
127	75	COMB1	Combination	0.940429572	-8.22E-16	53.71875	-7.83E-16	32.58787122	5.43E-03
127	39	COMB2	Combination	14.50071732	-1.25E-02	-112.6055181	1.84E-03	207.3521578	-2.30E-02
127	75	COMB2	Combination	-14.50071732	1.25E-02	198.5555181	-1.84E-03	259.3893964	-1.43E-02
127	39	COMB3	Combination	-16.00540463	0.012454405	198.5555181	-1.84E-03	-259.4927518	1.43E-02
127	75	COMB3	Combination	16.00540463	-0.012454405	-112.6055181	1.84E-03	-207.2488024	2.30E-02
127	39	COMB4	Combination	18.48577508	-1.56E-02	-146.6631476	2.30E-03	262.7583095	-2.55E-02
127	75	COMB4	Combination	-18.48577508	1.56E-02	242.2881476	-2.30E-03	320.6686333	-2.12E-02
127	39	COMB5	Combination	-19.64687737	1.56E-02	242.2881476	-2.30E-03	-320.7978276	2.12E-02
127	75	COMB5	Combination	19.64687737	-1.56E-02	-146.6631476	2.30E-03	-262.6291152	2.55E-02
127	39	COMB6	Combination	11.09146505	-9.34E-03	-87.99788855	1.38E-03	157.6549857	-1.53E-02
127	75	COMB6	Combination	-11.09146505	9.34E-03	145.3728885	-1.38E-03	192.40118	-0.012728669
127	39	COMB7	Combination	-11.78812642	9.34E-03	145.3728885	-1.38E-03	-192.4786965	1.27E-02
127	75	COMB7	Combination	11.78812642	-9.34E-03	-87.99788855	1.38E-03	-157.5774691	1.53E-02
128	40	COMB1	Combination	-1.11831552	1.37E-15	53.71875	-6.46E-16	-32.93168143	-5.19E-03
128	76	COMB1	Combination	1.11831552	-1.37E-15	53.71875	6.46E-16	32.93168143	5.19E-03
128	40	COMB2	Combination	30.81192149	-1.77E-02	-82.0152913	2.18E-03	161.2137815	-3.07E-02

128	76	COMB2	Combination	-30.81192149	1.77E-02	167.9652913	-2.18E-03	213.7570925	-0.022442129
128	40	COMB3	Combination	-32.60122632	0.017728245	167.9652913	-2.18E-03	-213.9044718	0.022442129
128	76	COMB3	Combination	32.60122632	-0.017728245	-82.0152913	2.18E-03	-161.0664022	3.07E-02
128	40	COMB4	Combination	38.74797647	-2.22E-02	-108.4253641	2.72E-03	205.1282576	-3.56E-02
128	76	COMB4	Combination	-38.74797647	2.22E-02	204.0503641	-2.72E-03	263.5853348	-3.08E-02
128	40	COMB5	Combination	-40.51845829	2.22E-02	204.0503641	-2.72E-03	-263.7695589	3.08E-02
128	76	COMB5	Combination	40.51845829	-2.22E-02	-108.4253641	2.72E-03	-204.9440335	3.56E-02
128	40	COMB6	Combination	23.24878588	-1.33E-02	-65.05521848	1.63E-03	123.0769546	-2.14E-02
128	76	COMB6	Combination	-23.24878588	1.33E-02	122.4302185	-1.63E-03	158.1512009	-1.85E-02
128	40	COMB7	Combination	-24.31107497	1.33E-02	122.4302185	-1.63E-03	-158.2617353	1.85E-02
128	76	COMB7	Combination	24.31107497	-1.33E-02	-65.05521848	1.63E-03	-122.9664201	2.14E-02
129	41	COMB1	Combination	-1.096490973	3.79E-16	53.71875	5.33E-15	-32.12875003	9.09E-03
129	77	COMB1	Combination	1.096490973	-3.79E-16	53.71875	-5.33E-15	32.12875003	-9.09E-03
129	41	COMB2	Combination	51.93103853	-2.20E-02	-37.7720887	2.14E-03	95.46976543	-2.58E-02
129	77	COMB2	Combination	-51.93103853	2.20E-02	123.7220887	-2.14E-03	146.7715007	-4.03E-02
129	41	COMB3	Combination	-53.68542409	2.20E-02	123.7220887	-2.14E-03	-146.8757655	4.03E-02
129	77	COMB3	Combination	53.68542409	-2.20E-02	-37.7720887	2.14E-03	-95.36550062	2.58E-02
129	41	COMB4	Combination	65.965144	-2.76E-02	-53.12136087	2.67E-03	122.7440952	-2.55E-02
129	77	COMB4	Combination	-65.965144	2.76E-02	148.7463609	-2.67E-03	180.0574874	-0.057147592
129	41	COMB5	Combination	-66.05543427	2.76E-02	148.7463609	-2.67E-03	-180.1878184	5.71E-02
129	77	COMB5	Combination	66.05543427	-2.76E-02	-53.12136087	2.67E-03	-122.6137642	2.55E-02
129	41	COMB6	Combination	39.5790864	-1.65E-02	-31.87281652	1.60E-03	73.64645712	-1.53E-02
129	77	COMB6	Combination	-39.5790864	1.65E-02	89.24781652	-1.60E-03	108.0344925	-3.43E-02
129	41	COMB7	Combination	-39.63326056	1.65E-02	89.24781652	-1.60E-03	-108.1126911	3.43E-02
129	77	COMB7	Combination	39.63326056	-1.65E-02	-31.87281652	1.60E-03	-73.56825852	1.53E-02
130	42	COMB1	Combination	14.72192661	-3.16E-15	21.9375	5.59E-15	-15.61263795	8.19E-02
130	78	COMB1	Combination	-14.72192661	3.16E-15	21.9375	-5.59E-15	15.61263795	-8.19E-02
130	42	COMB2	Combination	69.84442358	-2.59E-02	-10.80423225	3.99E-03	30.06331983	2.66E-02
130	78	COMB2	Combination	-69.84442358	2.59E-02	45.90423225	-3.99E-03	54.99937693	-0.104377899
130	42	COMB3	Combination	-46.289341	2.59E-02	45.90423225	-3.99E-03	-55.04354055	0.104377899
130	78	COMB3	Combination	46.289341	-2.59E-02	-10.80423225	3.99E-03	-30.0191562	-2.66E-02
130	42	COMB4	Combination	83.73981207	-3.24E-02	-19.41154031	4.98E-03	41.39427062	-1.87E-02
130	78	COMB4	Combination	-83.73981207	3.24E-02	51.47404031	-4.98E-03	64.93410032	-7.86E-02
130	42	COMB5	Combination	-61.42739364	3.24E-02	51.47404031	-4.98E-03	-64.98930485	7.86E-02
130	78	COMB5	Combination	61.42739364	-3.24E-02	-19.41154031	4.98E-03	-41.33906609	1.87E-02

130	42	COMB6	Combination	50.24388724	-1.94E-02	-11.64692419	2.99E-03	24.83656237	-0.011199691
130	78	COMB6	Combination	-50.24388724	1.94E-02	30.88442419	-2.99E-03	38.96046019	-4.71E-02
130	42	COMB7	Combination	-36.85643618	1.94E-02	30.88442419	-2.99E-03	-38.99358291	4.71E-02
130	78	COMB7	Combination	36.85643618	-1.94E-02	-11.64692419	2.99E-03	-24.80343965	1.12E-02
136	8	COMB1	Combination	-16.02026667	2.58E-02	97.23651049	-2.23E-03	-66.16529512	0.044491137
136	44	COMB1	Combination	16.02026667	-2.58E-02	98.51348951	2.23E-03	68.71925314	5.85E-02
136	8	COMB2	Combination	-21.59158603	8.22E-03	-29.60965258	-1.33E-03	169.6803407	5.97E-03
136	44	COMB2	Combination	21.59158603	-8.22E-03	186.2096526	1.33E-03	261.9582696	2.69E-02
136	8	COMB3	Combination	-4.040840639	3.30E-02	185.1880694	-2.24E-03	-275.5448129	6.52E-02
136	44	COMB3	Combination	4.040840639	-3.30E-02	-28.58806937	2.24E-03	-152.0074646	0.066722934
136	8	COMB4	Combination	-23.18664097	-2.68E-03	-58.53276237	1.62E-03	227.5268246	-1.49E-02
136	44	COMB4	Combination	23.18664097	2.68E-03	212.2827624	-1.62E-03	314.1042248	4.18E-03
136	8	COMB5	Combination	-1.248209229	2.83E-02	209.9643901	4.84E-04	-329.0046173	5.92E-02
136	44	COMB5	Combination	1.248209229	-2.83E-02	-56.21439007	-4.84E-04	-203.3529429	5.39E-02
136	8	COMB6	Combination	-13.91198458	-1.61E-03	-35.11965742	9.74E-04	136.5160948	-8.94E-03
136	44	COMB6	Combination	13.91198458	1.61E-03	127.3696574	-9.74E-04	188.4625349	2.51E-03
136	8	COMB7	Combination	-0.748925538	1.70E-02	125.978634	2.90E-04	-197.4027704	3.55E-02
136	44	COMB7	Combination	0.748925538	-1.70E-02	-33.72863404	-2.90E-04	-122.0117658	3.24E-02
137	9	COMB1	Combination	1.854180559	-3.05E-03	101.3259858	3.87E-02	-76.47709112	-8.55E-03
137	45	COMB1	Combination	-1.854180559	3.05E-03	94.42401419	-3.87E-02	62.67314789	-3.63E-03
137	9	COMB2	Combination	33.06651022	-2.43E-02	-13.45078812	3.16E-02	131.6678773	-5.92E-02
137	45	COMB2	Combination	-33.06651022	2.43E-02	170.0507881	-3.16E-02	235.3352752	-3.79E-02
137	9	COMB3	Combination	-30.09982133	1.94E-02	175.5723654	3.02E-02	-254.0312231	4.55E-02
137	45	COMB3	Combination	30.09982133	-1.94E-02	-18.97236541	-3.02E-02	-135.0582386	3.21E-02
137	9	COMB4	Combination	41.17884	-0.028949499	-39.61322808	1.42E-02	183.0528368	-0.069838
137	45	COMB4	Combination	-41.17884	0.028949499	193.3632281	-1.42E-02	282.9000755	-4.60E-02
137	9	COMB5	Combination	-37.77907443	2.57E-02	196.6657138	1.25E-02	-299.0710386	6.11E-02
137	45	COMB5	Combination	37.77907443	-2.57E-02	-42.91571384	-1.25E-02	-180.0918168	4.16E-02
137	9	COMB6	Combination	24.707304	-0.0173697	-23.76793685	8.53E-03	109.8317021	-4.19E-02
137	45	COMB6	Combination	-24.707304	0.0173697	116.0179368	-8.53E-03	169.7400453	-2.76E-02
137	9	COMB7	Combination	-22.66744466	1.54E-02	117.9994283	7.51E-03	-179.4426231	3.67E-02
137	45	COMB7	Combination	22.66744466	-1.54E-02	-25.7494283	-7.51E-03	-108.0550901	2.50E-02
138	10	COMB1	Combination	-0.893379772	3.53E-03	102.8121084	3.88E-02	-78.81864472	6.67E-03
138	46	COMB1	Combination	0.893379772	-3.53E-03	92.93789156	-3.88E-02	59.07021094	7.45E-03
138	10	COMB2	Combination	52.97498186	-2.78E-02	1.867339136	3.19E-02	101.4983494	-6.81E-02

138	46	COMB2	Combination	-52.97498186	2.78E-02	154.7326609	-3.19E-02	204.2322941	-4.31E-02
138	10	COMB3	Combination	-54.4043895	3.34E-02	162.6320344	3.01E-02	-227.6081809	7.87E-02
138	46	COMB3	Combination	54.4043895	-3.34E-02	-6.032034375	-3.01E-02	-109.7199566	5.50E-02
138	10	COMB4	Combination	66.15610904	-3.55E-02	-21.0636738	1.58E-02	146.3205385	-0.086020309
138	46	COMB4	Combination	-66.15610904	3.55E-02	174.8136738	-1.58E-02	245.4341567	-5.59E-02
138	10	COMB5	Combination	-68.06810517	4.11E-02	179.8921953	1.36E-02	-265.0626243	9.74E-02
138	46	COMB5	Combination	68.06810517	-4.11E-02	-26.14219525	-1.36E-02	-147.0061567	6.68E-02
138	10	COMB6	Combination	39.69366542	-2.13E-02	-12.63820428	9.48E-03	87.79232312	-5.16E-02
138	46	COMB6	Combination	-39.69366542	2.13E-02	104.8882043	-9.48E-03	147.260494	-3.35E-02
138	10	COMB7	Combination	-40.8408631	2.46E-02	107.9353172	8.15E-03	-159.0375746	5.85E-02
138	46	COMB7	Combination	40.8408631	-2.46E-02	-15.68531715	-8.15E-03	-88.20369402	4.01E-02
139	11	COMB1	Combination	-3.279847226	6.98E-03	104.1753388	4.90E-02	-81.95712428	1.78E-02
139	47	COMB1	Combination	3.279847226	-6.98E-03	91.57466121	-4.90E-02	56.75576913	1.01E-02
139	11	COMB2	Combination	87.24921539	-3.23E-02	28.16277461	0.040231464	46.93796678	-7.66E-02
139	47	COMB2	Combination	-87.24921539	3.23E-02	128.4372254	-0.040231464	153.6109348	-5.27E-02
139	11	COMB3	Combination	-92.49697096	4.35E-02	138.5177674	3.82E-02	-178.0693656	0.105047902
139	47	COMB3	Combination	92.49697096	-4.35E-02	18.08223255	-3.82E-02	-62.80170416	6.89E-02
139	11	COMB4	Combination	111.9620336	-5.10E-02	11.05013719	3.03E-03	80.00180268	-0.117894143
139	47	COMB4	Combination	-111.9620336	5.10E-02	142.6998628	-3.03E-03	183.2976485	-0.086212908
139	11	COMB5	Combination	-112.7206993	4.37E-02	148.9938782	5.01E-04	-201.2573628	0.109150002
139	47	COMB5	Combination	112.7206993	-4.37E-02	4.75612176	-5.01E-04	-87.21815011	6.57E-02
139	11	COMB6	Combination	67.17722019	-3.06E-02	6.630082316	1.82E-03	48.00108161	-7.07E-02
139	47	COMB6	Combination	-67.17722019	3.06E-02	85.61991768	-1.82E-03	109.9785891	-5.17E-02
139	11	COMB7	Combination	-67.63241957	2.62E-02	89.39632694	3.01E-04	-120.7544177	6.55E-02
139	47	COMB7	Combination	67.63241957	-2.62E-02	2.853673056	-3.01E-04	-52.33089007	3.94E-02
140	12	COMB1	Combination	25.91032588	-3.57E-02	65.0898901	0.112468071	-48.00112944	-5.70E-02
140	48	COMB1	Combination	-25.91032588	3.57E-02	57.1601099	-0.112468071	32.14156905	-8.59E-02
140	12	COMB2	Combination	118.5678177	-7.12E-02	26.29330662	9.15E-02	13.88275899	-0.147773653
140	48	COMB2	Combination	-118.5678177	7.12E-02	71.50669338	-9.15E-02	76.54401453	-0.136949365
140	12	COMB3	Combination	-77.11129629	1.40E-02	77.85051754	8.84E-02	-90.6845661	5.65E-02
140	48	COMB3	Combination	77.11129629	-1.40E-02	19.94948246	-8.84E-02	-25.11750406	-5.09E-04
140	12	COMB4	Combination	139.9725872	-6.51E-02	10.3354561	7.74E-02	33.8448602	-0.14546959
140	48	COMB4	Combination	-139.9725872	6.51E-02	69.9145439	-7.74E-02	85.31331539	-0.114938177
140	12	COMB5	Combination	-104.6263053	4.14E-02	74.78196975	7.35E-02	-96.86429616	0.109878205
140	48	COMB5	Combination	104.6263053	-4.14E-02	5.468030249	-7.35E-02	-41.76358285	0.055611824

140	12	COMB6	Combination	83.9835523	-3.91E-02	6.201273661	4.64E-02	20.30691612	-8.73E-02
140	48	COMB6	Combination	-83.9835523	3.91E-02	41.94872634	-4.64E-02	51.18798924	-6.90E-02
140	12	COMB7	Combination	-62.7757832	2.48E-02	44.86918185	4.41E-02	-58.11857769	6.59E-02
140	48	COMB7	Combination	62.7757832	-2.48E-02	3.280818149	-4.41E-02	-25.05814971	3.34E-02
141	44	COMB1	Combination	-10.3262291	1.55E-15	65.53125	6.61E-16	-42.2684246	-2.92E-02
141	80	COMB1	Combination	10.3262291	-1.55E-15	65.53125	-6.61E-16	42.2684246	2.92E-02
141	44	COMB2	Combination	-3.296842646	-8.35E-03	-120.1084257	1.35E-03	225.0112139	-3.59E-02
141	80	COMB2	Combination	3.296842646	8.35E-03	224.9584257	-1.35E-03	292.5890633	1.08E-02
141	44	COMB3	Combination	-13.22512391	8.35E-03	224.9584257	-1.35E-03	-292.6406933	-1.08E-02
141	80	COMB3	Combination	13.22512391	-8.35E-03	-120.1084257	1.35E-03	-224.9595839	3.59E-02
141	44	COMB4	Combination	-1.408693151	-1.04E-02	-161.9480322	1.68E-03	289.0372162	-3.01E-02
141	80	COMB4	Combination	1.408693151	1.04E-02	269.3855322	-1.68E-03	357.9631303	-1.22E-03
141	44	COMB5	Combination	-13.81904473	1.04E-02	269.3855322	-1.68E-03	-358.0276678	1.22E-03
141	80	COMB5	Combination	13.81904473	-1.04E-02	-161.9480322	1.68E-03	-288.9726787	3.01E-02
141	44	COMB6	Combination	-0.845215891	-6.26E-03	-97.16881929	1.01E-03	173.4223297	-1.81E-02
141	80	COMB6	Combination	0.845215891	6.26E-03	161.6313193	-1.01E-03	214.7778782	-7.29E-04
141	44	COMB7	Combination	-8.291426839	6.26E-03	161.6313193	-1.01E-03	-214.8166007	7.29E-04
141	80	COMB7	Combination	8.291426839	-6.26E-03	-97.16881929	1.01E-03	-173.3836072	1.81E-02
142	45	COMB1	Combination	-1.481207065	1.29E-15	65.53125	1.06E-15	-39.85659084	-5.34E-03
142	81	COMB1	Combination	1.481207065	-1.29E-15	65.53125	-1.06E-15	39.85659084	5.34E-03
142	45	COMB2	Combination	14.06809533	-1.51E-02	-103.8137935	1.86E-03	202.5245953	-2.69E-02
142	81	COMB2	Combination	-14.06809533	1.51E-02	208.6637935	-1.86E-03	266.1917852	-1.84E-02
142	45	COMB3	Combination	-16.43802663	1.51E-02	208.6637935	-1.86E-03	-266.2951407	1.84E-02
142	81	COMB3	Combination	16.43802663	-1.51E-02	-103.8137935	1.86E-03	-202.4212399	2.69E-02
142	45	COMB4	Combination	18.28250569	-1.89E-02	-141.5797419	2.32E-03	260.2982439	-3.07E-02
142	81	COMB4	Combination	-18.28250569	1.89E-02	249.0172419	-2.32E-03	325.5972318	-2.59E-02
142	45	COMB5	Combination	-19.85014675	1.89E-02	249.0172419	-2.32E-03	-325.726426	2.59E-02
142	81	COMB5	Combination	19.85014675	-1.89E-02	-141.5797419	2.32E-03	-260.1690496	3.07E-02
142	45	COMB6	Combination	10.96950341	-1.13E-02	-84.94784514	1.39E-03	156.1789463	-1.84E-02
142	81	COMB6	Combination	-10.96950341	1.13E-02	149.4103451	-1.39E-03	195.3583391	-1.56E-02
142	45	COMB7	Combination	-11.91008805	1.13E-02	149.4103451	-1.39E-03	-195.4358556	1.56E-02
142	81	COMB7	Combination	11.91008805	-1.13E-02	-84.94784514	1.39E-03	-156.1014298	1.84E-02
143	46	COMB1	Combination	-1.296269237	1.72E-15	65.53125	3.91E-15	-40.16703221	-3.28E-03
143	82	COMB1	Combination	1.296269237	-1.72E-15	65.53125	-3.91E-15	40.16703221	3.28E-03
143	46	COMB2	Combination	30.66955851	-2.13E-02	-73.13963713	2.19E-03	156.2870196	-3.46E-02

143	82	COMB2	Combination	-30.66955851	2.13E-02	177.9896371	-2.19E-03	220.4068918	-0.02934362
143	46	COMB3	Combination	-32.74358929	2.13E-02	177.9896371	-2.19E-03	-220.5542711	2.93E-02
143	82	COMB3	Combination	32.74358929	-2.13E-02	-73.13963713	2.19E-03	-156.1396403	3.46E-02
143	46	COMB4	Combination	38.79009946	-2.66E-02	-103.2370464	2.74E-03	202.5901407	-4.11E-02
143	82	COMB4	Combination	-38.79009946	2.66E-02	210.6745464	-2.74E-03	268.2772485	-3.88E-02
143	46	COMB5	Combination	-40.4763353	2.66E-02	210.6745464	-2.74E-03	-268.4614726	3.88E-02
143	82	COMB5	Combination	40.4763353	-2.66E-02	-103.2370464	2.74E-03	-202.4059166	4.11E-02
143	46	COMB6	Combination	23.27405968	-1.60E-02	-61.94222785	1.65E-03	121.5540844	-2.47E-02
143	82	COMB6	Combination	-23.27405968	1.60E-02	126.4047279	-1.65E-03	160.9663491	-2.33E-02
143	46	COMB7	Combination	-24.28580118	0.015981922	126.4047279	-1.65E-03	-161.0768836	2.33E-02
143	82	COMB7	Combination	24.28580118	-0.015981922	-61.94222785	1.65E-03	-121.44355	2.47E-02
144	47	COMB1	Combination	-3.150364427	7.10E-16	65.53125	3.41E-15	-39.6144035	5.96E-03
144	83	COMB1	Combination	3.150364427	-7.10E-16	65.53125	-3.41E-15	39.6144035	-5.96E-03
144	47	COMB2	Combination	50.28793977	-2.64E-02	-28.7429801	2.30E-03	90.11257975	-3.49E-02
144	83	COMB2	Combination	-50.28793977	2.64E-02	133.5929801	-2.30E-03	153.3913605	-4.44E-02
144	47	COMB3	Combination	-55.32852285	2.64E-02	133.5929801	-2.30E-03	-153.4956254	4.44E-02
144	83	COMB3	Combination	55.32852285	-2.64E-02	-28.7429801	2.30E-03	-90.00831494	3.49E-02
144	47	COMB4	Combination	64.93907832	-3.30E-02	-47.74122512	2.87E-03	119.4645043	-4.08E-02
144	83	COMB4	Combination	-64.93907832	3.30E-02	155.1787251	-2.87E-03	184.9154211	-5.83E-02
144	47	COMB5	Combination	-67.08149996	3.30E-02	155.1787251	-2.87E-03	-185.0457521	5.83E-02
144	83	COMB5	Combination	67.08149996	-3.30E-02	-47.74122512	2.87E-03	-119.3341733	0.040754428
144	47	COMB6	Combination	38.96344699	-1.98E-02	-28.64473507	1.72E-03	71.67870259	-0.024452657
144	83	COMB6	Combination	-38.96344699	1.98E-02	93.10723507	-1.72E-03	110.9492526	-3.50E-02
144	47	COMB7	Combination	-40.24889998	1.98E-02	93.10723507	-1.72E-03	-111.0274512	3.50E-02
144	83	COMB7	Combination	40.24889998	-1.98E-02	-28.64473507	1.72E-03	-71.60050398	2.45E-02
145	48	COMB1	Combination	20.75964801	-1.45E-15	36.28125	4.58E-15	-22.88785005	5.12E-02
145	84	COMB1	Combination	-20.75964801	1.45E-15	36.28125	-4.58E-15	22.88785005	-5.12E-02
145	48	COMB2	Combination	74.6746007	-3.06E-02	0.499521494	3.37E-03	24.50001953	-4.91E-03
145	84	COMB2	Combination	-74.6746007	3.06E-02	57.55047851	-3.37E-03	61.07641599	-8.68E-02
145	48	COMB3	Combination	-41.45916388	3.06E-02	57.55047851	-3.37E-03	-61.12057961	0.08682058
145	84	COMB3	Combination	41.45916388	-3.06E-02	0.499521494	3.37E-03	-24.45585591	4.91E-03
145	48	COMB4	Combination	86.31680175	-3.82E-02	-11.18809813	4.21E-03	38.27210599	-3.78E-02
145	84	COMB4	Combination	-86.31680175	3.82E-02	60.12559813	-4.21E-03	68.6984384	-0.076810974
145	48	COMB5	Combination	-58.85040396	3.82E-02	60.12559813	-4.21E-03	-68.75364293	7.68E-02
145	84	COMB5	Combination	58.85040396	-3.82E-02	-11.18809813	4.21E-03	-38.21690146	0.037848318

145	48	COMB6	Combination	51.79008105	-2.29E-02	-6.71285888	2.53E-03	22.9632636	-2.27E-02
145	84	COMB6	Combination	-51.79008105	2.29E-02	36.07535888	-2.53E-03	41.21906304	-4.61E-02
145	48	COMB7	Combination	-35.31024238	2.29E-02	36.07535888	-2.53E-03	-41.25218576	4.61E-02
145	84	COMB7	Combination	35.31024238	-2.29E-02	-6.71285888	2.53E-03	-22.93014088	2.27E-02

Analysis result of RC frame with EQ force in Y direction:

Table of element forces-Frames

Frame	Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3
Text	Text	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
1	1	COMB1	Combination	5.887416403	4.470965719	791.8595834	6.833031639	8.780756772	2.73E-02
1	2	COMB1	Combination	5.887416403	4.470965719	766.5470834	-13.2863141	17.71261704	-2.73E-02
1	1	COMB2	Combination	5.468500051	66.06995617	268.4093979	167.6788659	20.00644972	0.644115891
1	2	COMB2	Combination	5.468500051	66.06995617	248.1593979	129.6359368	4.601800513	0.644115891
1	1	COMB3	Combination	14.8883663	73.22350132	998.5659355	178.6117165	34.05566055	0.600388783
1	2	COMB3	Combination	-14.8883663	73.22350132	978.3159355	150.8940394	32.94198778	0.600388783
1	1	COMB4	Combination	7.712996451	83.35087753	223.0796299	210.768212	26.31780967	0.791439683
1	2	COMB4	Combination	7.712996451	83.35087753	197.7671299	164.3107368	8.390674358	0.791439683
1	1	COMB5	Combination	17.73308648	90.76594434	1135.775302	-222.095016	41.25982816	-0.76419116
1	2	COMB5	Combination	17.73308648	90.76594434	1110.462802	186.3517335	38.53906101	0.76419116
1	1	COMB6	Combination	9.717014457	84.83389089	48.69135648	213.0335728	29.30621337	0.785989978
1	2	COMB6	Combination	9.717014457	84.83389089	63.87885648	168.7189362	14.42035169	0.785989978
1	1	COMB7	Combination	15.72906848	89.28293097	864.0043155	219.8296552	38.27142446	0.769640864
1	2	COMB7	Combination	15.72906848	89.28293097	848.8168155	181.9435341	32.50938368	0.769640864
2	2	COMB1	Combination	18.27585335	14.033118	625.0503391	25.44421509	33.63643003	-2.83E-02
2	3	COMB1	Combination	18.27585335	-14.033118	605.3628391	23.67169792	30.32905669	2.83E-02
2	2	COMB2	Combination	7.223018978	37.79275458	251.0495723	58.32836436	15.06070873	2.64E-02
2	3	COMB2	Combination	7.223018978	37.79275458	235.2995723	73.94627667	10.21985769	-2.64E-02
2	2	COMB3	Combination	22.01834638	60.24574338	749.0309702	-99.0391085	38.75757932	-7.17E-02
2	3	COMB3	Combination	22.01834638	60.24574338	733.2809702	111.8209933	38.30663302	7.17E-02
2	2	COMB4	Combination	6.243277307	49.67040638	222.4689905	77.29007128	13.73602577	0.046663623
2	3	COMB4	Combination	6.243277307	49.67040638	202.7814905	96.55635104	8.115444809	0.046663623
2	2	COMB5	Combination	24.73743656	72.87771607	844.9457377	119.4192698	43.357114	-7.60E-02
2	3	COMB5	Combination	24.73743656	72.87771607	825.2582377	135.6527365	43.22391397	7.60E-02

2	2	COMB6	Combination	4.71E-02	54.31186832	8.986044813	85.71591098	2.317397813	0.052526364
2	3	COMB6	Combination	-4.71E-02	54.31186832	2.826455187	104.3756281	2.152426947	0.052526364
2	2	COMB7	Combination	18.54129379	68.23625414	631.4627921	110.9934301	31.93848604	-7.01E-02
2	3	COMB7	Combination	18.54129379	68.23625414	619.6502921	127.8334594	32.95604221	7.01E-02
3	3	COMB1	Combination	16.81041199	13.61452457	457.356225	23.51048424	28.78438433	0.004405796
3	4	COMB1	Combination	16.81041199	13.61452457	-437.668725	24.14035175	30.05205765	0.004405796
3	3	COMB2	Combination	6.086272094	38.22765724	212.9129293	63.96451084	10.90850111	0.121613071
3	4	COMB2	Combination	6.086272094	38.22765724	197.1629293	69.83228951	10.39345122	0.121613071
3	3	COMB3	Combination	20.8103871	60.01089656	518.8570306	101.5812856	35.14651382	0.128662345
3	4	COMB3	Combination	-20.8103871	60.01089656	503.1070306	108.4568523	37.68984102	0.128662345
3	3	COMB4	Combination	4.894041736	50.25300857	197.2900328	84.19081882	9.026601449	0.154065864
3	4	COMB4	Combination	4.894041736	50.25300857	177.6025328	91.69471117	8.102544629	0.154065864
3	3	COMB5	Combination	23.29918549	72.54518368	579.7201594	122.7414268	39.32411734	0.158778406
3	4	COMB5	Combination	23.29918549	72.54518368	560.0326594	131.1667161	42.22303188	0.158778406
3	3	COMB6	Combination	0.744603709	54.71144359	41.88799434	91.90094041	0.643542308	0.155008373
3	4	COMB6	Combination	0.744603709	54.71144359	30.07549434	99.58911216	1.962570673	0.155008373
3	3	COMB7	Combination	17.66054005	68.08674866	424.318121	115.0313052	29.65397358	0.157835898
3	4	COMB7	Combination	17.66054005	68.08674866	-412.505621	123.2723151	32.15791658	0.157835898
4	4	COMB1	Combination	17.23658507	14.36801322	287.4342073	25.08145657	30.60885349	-1.51E-02
4	5	COMB1	Combination	17.23658507	14.36801322	267.7467073	25.20658968	29.71919427	1.51E-02
4	4	COMB2	Combination	8.068297789	27.04112621	157.4889019	39.69518169	16.08595639	0.155787201
4	5	COMB2	Combination	8.068297789	27.04112621	141.7389019	54.94876006	12.15308587	0.155787201
4	4	COMB3	Combination	19.51023833	50.02994736	302.4058298	-79.8255122	32.88820919	0.179930736
4	5	COMB3	Combination	19.51023833	50.02994736	286.6558298	95.27930355	35.39762495	0.179930736
4	4	COMB4	Combination	7.399859144	36.35182497	151.1337757	54.15516364	15.22301522	0.195947573
4	5	COMB4	Combination	7.399859144	36.35182497	131.4462757	73.07622374	10.67649179	0.195947573
4	4	COMB5	Combination	21.70228482	59.987017	332.2799355	95.24570372	36.22583122	0.223699848
4	5	COMB5	Combination	21.70228482	-59.987017	312.5924355	114.7088558	39.73216564	0.223699848
4	4	COMB6	Combination	1.579430352	41.07886337	54.45103346	62.37327165	4.933245928	0.201498028
4	5	COMB6	Combination	1.579430352	41.07886337	42.63853346	81.40275015	0.594760303	0.201498028
4	4	COMB7	Combination	15.88185603	55.25997859	235.5971933	87.02759571	25.93606194	0.218149393
4	5	COMB7	Combination	15.88185603	55.25997859	223.7846933	106.3823294	29.65043415	0.218149393
5	5	COMB1	Combination	20.46003242	15.16879072	115.9247639	-25.5906865	33.25953371	-5.09E-02
5	6	COMB1	Combination	20.46003242	15.16879072	96.23726391	27.50008101	38.35057977	5.09E-02
5	5	COMB2	Combination	13.56211037	4.624718611	72.64085393	0.664004532	23.47527698	3.98E-02

5	6	COMB2	Combination	13.56211037	4.624718611	56.89085393	15.52251061	23.9921093	-3.98E-02
5	5	COMB3	Combination	19.17394151	28.89478376	112.8387683	41.60910293	29.73997696	0.121185332
5	6	COMB3	Combination	19.17394151	28.89478376	97.08876832	59.52264023	37.36881833	0.121185332
5	5	COMB4	Combination	12.81083214	9.271495228	68.92799998	6.226959572	23.13410751	8.96E-02
5	6	COMB4	Combination	12.81083214	9.271495228	49.24049998	26.22327373	21.703805	-8.96E-02
5	5	COMB5	Combination	19.82562107	32.62788274	119.175393	46.61442475	30.96498247	-0.11161587
5	6	COMB5	Combination	19.82562107	32.62788274	99.48789297	67.58316482	38.42469128	0.11161587
5	5	COMB6	Combination	6.283541501	13.94277273	31.30732139	14.30445261	12.31428951	9.40E-02
5	6	COMB6	Combination	6.283541501	13.94277273	19.49482139	34.49525194	9.678105743	-9.40E-02
5	5	COMB7	Combination	13.29833043	27.95660523	81.55471438	38.53693172	20.14516448	0.107207497
5	6	COMB7	Combination	13.29833043	27.95660523	69.74221438	-59.3111866	26.39899203	0.107207497
6	7	COMB1	Combination	7.573603204	7.75E-02	1199.786298	0.234429899	11.2963349	1.52E-02
6	8	COMB1	Combination	7.573603204	-7.75E-02	1174.473798	0.114238335	22.78487952	-1.52E-02
6	7	COMB2	Combination	4.454497809	84.37878262	943.745351	194.9634879	18.82991509	0.595370251
6	8	COMB2	Combination	4.454497809	84.37878262	-923.495351	184.7410339	1.215325055	0.595370251
6	7	COMB3	Combination	16.57226294	84.50275355	975.9127256	195.3385758	36.90405092	-0.57103854
6	8	COMB3	Combination	16.57226294	84.50275355	955.6627256	184.9238152	37.67113229	0.57103854
6	7	COMB4	Combination	7.320127115	105.5288688	963.5719385	243.8094235	26.15258036	0.736516149
6	8	COMB4	Combination	7.320127115	105.5288688	938.2594385	231.070486	6.787991657	0.736516149
6	7	COMB5	Combination	18.96332382	105.5730514	1003.781157	244.0681562	43.51487715	0.721494839
6	8	COMB5	Combination	18.96332382	105.5730514	978.4686568	231.0105753	41.82008002	0.721494839
6	7	COMB6	Combination	9.648766455	105.5377053	570.1013194	243.86117	29.62503972	0.733511887
6	8	COMB6	Combination	9.648766455	105.5377053	554.9138194	231.0585039	13.79440933	0.733511887
6	7	COMB7	Combination	16.63468448	105.5642149	610.3105378	244.0164096	40.04241779	0.724499101
6	8	COMB7	Combination	16.63468448	105.5642149	595.1230378	231.0225574	34.81366235	0.724499101
7	8	COMB1	Combination	23.59752847	8.34E-02	937.6525433	0.286378093	43.38017319	-1.45E-02
7	9	COMB1	Combination	23.59752847	-8.34E-02	917.9650433	0.578106398	39.21117644	1.45E-02
7	8	COMB2	Combination	11.06877861	93.67994842	730.3076721	162.9576351	22.1658419	3.71E-02
7	9	COMB2	Combination	11.06877861	93.67994842	714.5576721	164.9221844	16.57488323	-3.71E-02
7	8	COMB3	Combination	26.68726694	93.81330993	769.9363973	162.4994301	47.2424352	-6.03E-02
7	9	COMB3	Combination	26.68726694	93.81330993	754.1863973	165.8471547	46.16299908	6.03E-02
7	8	COMB4	Combination	8.279243486	117.3014866	739.473132	203.9599154	17.54986464	5.34E-02
7	9	COMB4	Combination	8.279243486	117.3014866	-719.785632	206.5952878	11.42748756	-5.34E-02
7	8	COMB5	Combination	27.80235389	117.0650863	789.0090385	202.8614161	48.89560626	-6.82E-02
7	9	COMB5	Combination	27.80235389	117.0650863	769.3215385	-206.866386	48.41263237	6.82E-02

7	8	COMB6	Combination	1.06292401	117.2542066	433.7766979	203.7402155	4.260770458	5.64E-02
7	9	COMB6	Combination	-1.06292401	117.2542066	421.9641979	206.6495075	0.540536423	-5.64E-02
7	8	COMB7	Combination	20.58603442	117.1123664	483.3126044	-203.081116	35.60651208	0.065264077
7	9	COMB7	Combination	20.58603442	117.1123664	471.5001044	206.8121664	36.44460838	0.065264077
8	9	COMB1	Combination	21.74364079	1.552966631	681.1520943	2.684845188	37.2657412	-3.32E-03
8	10	COMB1	Combination	21.74364079	1.552966631	661.4645943	2.750538021	38.83700158	3.32E-03
8	9	COMB2	Combination	9.560592142	80.47266388	530.0371292	137.643492	16.88965498	0.113529414
8	10	COMB2	Combination	9.560592142	80.47266388	514.2871292	144.0108316	16.57241752	0.113529414
8	9	COMB3	Combination	25.22923313	82.95741049	559.8062217	141.9392443	42.73553095	0.118840946
8	10	COMB3	Combination	25.22923313	82.95741049	544.0562217	148.4116924	45.566785	0.118840946
8	9	COMB4	Combination	6.548079698	101.0896376	532.23851	172.9162372	11.93528574	0.143255113
8	10	COMB4	Combination	6.548079698	101.0896376	-512.55101	180.8974944	10.9829932	0.143255113
8	9	COMB5	Combination	26.13388093	103.1979554	569.4498756	176.5621832	44.2426307	0.147207836
8	10	COMB5	Combination	26.13388093	103.1979554	549.7623756	184.6306606	47.22595255	0.147207836
8	9	COMB6	Combination	1.17E-02	101.5113012	311.9008329	173.6454264	0.69970245	0.144045658
8	10	COMB6	Combination	-1.17E-02	101.5113012	300.0883329	181.6441277	0.658795947	0.144045658
8	9	COMB7	Combination	19.5974888	102.7762918	349.1121985	-175.832994	33.00704741	0.146417291
8	10	COMB7	Combination	-19.5974888	102.7762918	337.2996985	183.8840273	35.5841634	0.146417291
9	10	COMB1	Combination	22.63733905	1.938539859	424.3275215	3.233157058	39.98144591	-8.65E-03
9	11	COMB1	Combination	22.63733905	1.938539859	404.6400215	3.551732449	39.24924077	8.65E-03
9	10	COMB2	Combination	11.93780721	-62.8356512	328.8293326	105.2424826	22.82472723	0.151535216
9	11	COMB2	Combination	11.93780721	62.8356512	313.0793326	114.6822966	18.957598	0.151535216
9	10	COMB3	Combination	24.28193528	65.93731498	350.0947018	110.4155339	41.14558623	0.165377847
9	11	COMB3	Combination	24.28193528	65.93731498	334.3447018	120.3650685	43.84118724	0.165377847
9	10	COMB4	Combination	9.582099464	79.24829476	324.1127531	132.7149961	18.81594843	0.190900632
9	11	COMB4	Combination	9.582099464	79.24829476	304.4252531	144.6540355	14.72139969	0.190900632
9	10	COMB5	Combination	25.01225955	81.71791297	350.6944646	136.8575245	41.71702218	0.205240696
9	11	COMB5	Combination	25.01225955	81.71791297	331.0069646	149.1551709	45.82588624	0.205240696
9	10	COMB6	Combination	2.663227661	-79.7422184	189.1513096	133.5435018	6.709354309	0.193768645
9	11	COMB6	Combination	2.663227661	79.7422184	177.3388096	145.5542626	2.611942505	0.193768645
9	10	COMB7	Combination	18.09338775	81.22398933	215.733021	136.0290188	29.61042806	0.202372683
9	11	COMB7	Combination	18.09338775	81.22398933	-203.920521	148.2549438	33.71642905	0.202372683
10	11	COMB1	Combination	25.91673698	1.80164469	167.1405599	2.975193307	42.7076737	-3.00E-02
10	12	COMB1	Combination	25.91673698	-1.80164469	147.4530599	3.330563109	48.00090574	3.00E-02
10	11	COMB2	Combination	17.45484421	32.37541922	129.0183277	50.3498118	30.33703581	0.053891724

10	12	COMB2	Combination	17.45484421	32.37541922	113.2683277	62.96415548	30.75491894	0.053891724
10	11	COMB3	Combination	24.01193496	35.25805073	138.4065681	55.11012109	37.99524211	-0.10181677
10	12	COMB3	Combination	24.01193496	35.25805073	122.6565681	68.29305645	46.04653024	0.10181677
10	11	COMB4	Combination	13.57717575	-41.0127092	118.0263214	63.90479673	25.56763145	8.96E-02
10	12	COMB4	Combination	13.57717575	41.0127092	98.33882138	79.63968546	21.95248366	-8.96E-02
10	11	COMB5	Combination	21.77353918	43.52912824	129.761622	67.92011939	35.14038934	0.104997335
10	12	COMB5	Combination	21.77353918	43.52912824	-110.074122	84.43182946	41.06699779	0.104997335
10	11	COMB6	Combination	6.507032763	41.51599301	68.4687327	64.70786126	13.4260273	9.27E-02
10	12	COMB6	Combination	6.507032763	41.51599301	-56.6562327	80.59811426	9.348587373	-9.27E-02
10	11	COMB7	Combination	14.70339619	43.02584443	80.20403333	67.11705485	22.99878518	0.101925524
10	12	COMB7	Combination	14.70339619	43.02584443	68.39153333	83.47340066	28.4631015	0.101925524
11	13	COMB1	Combination	7.577613538	0.100441535	1207.589083	0.188305385	11.30247222	5.10E-03
11	14	COMB1	Combination	7.577613538	0.100441535	1182.276583	0.263681524	22.7967887	-5.10E-03
11	13	COMB2	Combination	-4.51280395	82.81226827	914.6967583	192.686741	18.99748512	0.581598742
11	14	COMB2	Combination	4.51280395	82.81226827	894.4467583	179.9684662	-1.31013266	0.581598742
11	13	COMB3	Combination	16.63698561	82.97297472	1017.445775	192.9880296	37.08144068	0.573432935
11	14	COMB3	Combination	16.63698561	82.97297472	997.1957745	180.3903566	37.78499457	0.573432935
11	13	COMB4	Combination	7.394551075	103.5317021	923.1941541	240.890231	26.36441758	0.724407922
11	14	COMB4	Combination	7.394551075	103.5317021	897.8816541	225.0024284	6.911062255	0.724407922
11	13	COMB5	Combination	19.04268588	103.6998517	1051.630424	241.2032324	43.73423966	0.719381674
11	14	COMB5	Combination	19.04268588	103.6998517	1026.317924	225.4461001	41.95784679	0.719381674
11	13	COMB6	Combination	9.724178035	-103.565332	528.2292384	240.9528313	-29.838382	0.723402672
11	14	COMB6	Combination	9.724178035	103.565332	513.0417384	225.0911627	13.92041916	0.723402672
11	13	COMB7	Combination	16.71305892	103.6662217	656.6655087	241.1406321	40.26027524	0.720386924
11	14	COMB7	Combination	16.71305892	103.6662217	641.4780087	225.3573658	34.94848988	0.720386924
12	14	COMB1	Combination	23.61592307	0.103788701	948.64733	0.126413317	43.41022742	-4.83E-03
12	15	COMB1	Combination	23.61592307	0.103788701	-928.95983	0.236847139	39.24550331	4.83E-03
12	14	COMB2	Combination	11.06474065	89.61522871	725.2656252	154.093662	22.16659539	4.45E-02
12	15	COMB2	Combination	11.06474065	89.61522871	709.5156252	159.5596385	16.55999689	-4.45E-02
12	14	COMB3	Combination	26.72073626	89.44916679	792.5701028	153.8914007	47.28976849	-5.23E-02
12	15	COMB3	Combination	26.72073626	89.44916679	776.8201028	159.1806831	46.23280841	5.23E-02
12	14	COMB4	Combination	8.266330096	111.9932981	728.9358268	192.5810714	17.53827964	5.81E-02
12	15	COMB4	Combination	8.266330096	111.9932981	709.2483268	199.3954721	11.3938757	-5.81E-02
12	14	COMB5	Combination	27.83632461	111.8371962	813.0664239	-192.400257	48.94224603	-6.30E-02
12	15	COMB5	Combination	27.83632461	111.8371962	793.3789239	199.0299299	48.4848901	6.30E-02

12	14	COMB6	Combination	1.045799156	111.9620777	420.5353767	192.5449085	4.242174508	5.90E-02
12	15	COMB6	Combination	1.045799156	111.9620777	408.7228767	199.3223636	0.581877463	-5.90E-02
12	14	COMB7	Combination	20.61579367	111.8684166	504.6659737	192.4364198	35.64614089	-6.20E-02
12	15	COMB7	Combination	20.61579367	111.8684166	492.8534737	199.1030383	36.50913694	6.20E-02
13	15	COMB1	Combination	21.77206476	-8.64E-02	690.5863752	0.147411144	37.31291089	-1.15E-03
13	16	COMB1	Combination	21.77206476	8.64E-02	670.8988752	0.155059677	38.88931576	1.15E-03
13	15	COMB2	Combination	9.568934228	80.27857439	531.8940324	137.3785607	16.90808398	0.111579829
13	16	COMB2	Combination	9.568934228	80.27857439	516.1440324	143.5964497	16.58318582	0.111579829
13	15	COMB3	Combination	25.26636938	80.14030202	573.044168	137.1427028	42.79257345	0.113423725
13	16	COMB3	Combination	25.26636938	80.14030202	-557.294168	143.3483542	45.6397194	0.113423725
13	15	COMB4	Combination	6.545566364	100.3269434	531.1613114	171.6861852	11.93685689	0.139943247
13	16	COMB4	Combination	6.545566364	100.3269434	511.4738114	179.4581166	10.97262538	0.139943247
13	15	COMB5	Combination	26.16736031	100.1966521	582.5989808	171.4653941	44.29246873	0.141311196
13	16	COMB5	Combination	26.16736031	100.1966521	562.9114808	179.2228883	47.29329236	0.141311196
13	15	COMB6	Combination	2.98E-03	100.3008851	308.4092529	171.642027	0.690991767	0.140216837
13	16	COMB6	Combination	-2.98E-03	100.3008851	296.5967529	179.411071	0.680558166	0.140216837
13	15	COMB7	Combination	19.62477497	100.2227104	359.8469224	171.5095523	33.0466036	0.141037606
13	16	COMB7	Combination	19.62477497	100.2227104	348.0344224	-179.269934	35.64010881	0.141037606
14	16	COMB1	Combination	22.67184317	-6.22E-02	431.0455096	0.13127429	40.04207419	-3.11E-03
14	17	COMB1	Combination	22.67184317	6.22E-02	411.3580096	8.66E-02	39.3093769	3.11E-03
14	16	COMB2	Combination	11.96069168	-63.1733008	335.4125581	105.369836	22.87702248	0.151885237
14	17	COMB2	Combination	11.96069168	63.1733008	319.6625581	115.7367168	18.9853984	0.151885237
14	16	COMB3	Combination	24.31425739	63.07372066	354.2602572	105.1597972	41.19029622	0.156864482
14	17	COMB3	Combination	24.31425739	63.07372066	338.5102572	115.5982251	43.90960465	0.156864482
14	16	COMB4	Combination	9.594804624	78.95202562	330.0979702	131.6872232	18.85352166	0.19042001
14	17	COMB4	Combination	9.594804624	78.95202562	310.4104702	144.6448665	14.72829453	-0.19042001
14	16	COMB5	Combination	25.03676177	78.8567512	353.657594	131.4748183	41.74511384	0.195517139
14	17	COMB5	Combination	25.03676177	-78.8567512	-333.970094	144.5238109	45.88355234	0.195517139
14	16	COMB6	Combination	2.668491346	78.93297074	193.3468573	131.6447422	6.733794561	0.191439436
14	17	COMB6	Combination	2.668491346	78.93297074	181.5343573	144.6206553	2.605925152	0.191439436
14	16	COMB7	Combination	18.11044849	78.87580609	216.9064812	131.5172993	29.62538674	0.194497713
14	17	COMB7	Combination	18.11044849	78.87580609	205.0939812	-144.548022	33.76118297	0.194497713
15	17	COMB1	Combination	25.97364655	0.410534796	169.831043	0.664725741	42.79342866	-9.89E-03
15	18	COMB1	Combination	25.97364655	0.410534796	-150.143543	0.772146045	48.11433426	9.89E-03
15	17	COMB2	Combination	17.51635127	35.12301246	133.4230338	54.71686548	30.43834607	6.94E-02

15	18	COMB2	Combination	17.51635127	35.12301246	117.6730338	68.21367811	30.86888337	-6.94E-02
15	17	COMB3	Combination	24.04148321	34.46615678	138.306635	-53.6533043	38.03113979	0.085187118
15	18	COMB3	Combination	24.04148321	34.46615678	-122.556635	66.97824444	46.11405145	0.085187118
15	17	COMB4	Combination	13.6308727	-43.8049685	123.0256983	68.22037648	25.65825107	9.42E-02
15	18	COMB4	Combination	-13.6308727	43.8049685	103.3381983	85.09701325	22.04980338	-9.42E-02
15	17	COMB5	Combination	21.78728763	43.18149305	129.1301997	67.24233575	35.14924323	-9.90E-02
15	18	COMB5	Combination	21.78728763	43.18149305	109.4426997	83.89288993	41.10626349	9.90E-02
15	17	COMB6	Combination	6.547240635	43.68027341	72.59451866	68.02476833	13.49675221	9.51E-02
15	18	COMB6	Combination	6.547240635	43.68027341	60.78201866	84.85618859	9.418590009	-9.51E-02
15	17	COMB7	Combination	14.70365557	43.30618814	78.69902011	67.43794389	22.98774437	-9.80E-02
15	18	COMB7	Combination	14.70365557	43.30618814	66.88652011	-84.1337146	28.47505011	9.80E-02
31	37	COMB1	Combination	2.348942997	5.832708022	1105.65514	8.931252809	3.397743436	1.82E-02
31	38	COMB1	Combination	2.348942997	5.832708022	-1077.81139	17.31593329	7.172500051	-1.82E-02
31	37	COMB2	Combination	18.16891333	69.18389754	526.5032583	178.0978888	42.66448781	0.377148262
31	38	COMB2	Combination	18.16891333	69.18389754	504.2282583	133.2296502	39.09562219	0.377148262
31	37	COMB3	Combination	14.41060454	78.51623037	1242.544966	192.3878933	37.22809831	0.348050459
31	38	COMB3	Combination	14.41060454	78.51623037	1220.269966	160.9351434	27.61962211	0.348050459
31	37	COMB4	Combination	22.37936259	87.87547388	475.8521621	224.7625393	-52.8517894	0.462345153
31	38	COMB4	Combination	22.37936259	87.87547388	448.0084121	170.6770931	47.85534224	0.462345153
31	37	COMB5	Combination	18.34503475	96.74968601	1370.904296	238.3446882	47.01394325	0.444153247
31	38	COMB5	Combination	18.34503475	96.74968601	1343.060546	197.0288989	35.53871314	0.444153247
31	37	COMB6	Combination	21.57249702	89.65031631	106.5008704	227.4789691	51.68422017	0.458706772
31	38	COMB6	Combination	21.57249702	89.65031631	89.79462037	175.9474543	45.39201642	0.458706772
31	37	COMB7	Combination	19.15190032	94.97484358	1001.553005	235.6282584	48.18151248	0.447791629
31	38	COMB7	Combination	19.15190032	94.97484358	984.8467547	191.7585377	38.00203896	0.447791629
32	38	COMB1	Combination	6.723213287	18.0762371	861.6644385	32.81594242	13.11986587	-2.08E-02
32	39	COMB1	Combination	6.723213287	-18.0762371	840.0081885	30.45088742	10.41138064	2.08E-02
32	38	COMB2	Combination	23.35811083	36.34496985	445.9813571	54.55140376	41.71708734	-4.10E-02
32	39	COMB2	Combination	23.35811083	36.34496985	428.6563571	72.65599072	40.03630056	4.10E-02
32	38	COMB3	Combination	12.60096957	65.26694921	932.6817444	107.0569116	20.72530195	7.65E-03
32	39	COMB3	Combination	12.60096957	65.26694921	915.3567444	121.3774106	23.37809154	-7.65E-03
32	38	COMB4	Combination	28.32656659	49.81240557	411.2443361	76.106023	50.38945607	0.041079158
32	39	COMB4	Combination	28.32656659	49.81240557	389.5880861	98.2373965	48.75352699	0.041079158
32	38	COMB5	Combination	16.6222839	77.20249326	1019.61982	125.9043712	27.66353053	1.97E-02
32	39	COMB5	Combination	-16.6222839	77.20249326	997.9635704	144.3043551	30.51446313	-1.97E-02

32	38	COMB6	Combination	25.98571005	55.29042311	125.0715048	86.06569265	45.84427096	-3.68E-02
32	39	COMB6	Combination	25.98571005	55.29042311	112.0777548	107.4507882	45.10571422	3.68E-02
32	38	COMB7	Combination	18.96314044	71.72447572	733.4469891	115.9447016	32.20871564	2.40E-02
32	39	COMB7	Combination	18.96314044	71.72447572	720.4532391	135.0909634	34.1622759	-2.40E-02
33	39	COMB1	Combination	4.318517954	17.49133498	622.6826359	30.22323555	7.305287439	-1.14E-04
33	40	COMB1	Combination	4.318517954	17.49133498	601.0263859	-30.9964369	7.809525401	1.14E-04
33	39	COMB2	Combination	18.90106756	36.93860251	350.1171621	61.23950966	31.93506535	0.059998341
33	40	COMB2	Combination	18.90106756	36.93860251	332.7921621	68.04559914	34.21867113	0.059998341
33	39	COMB3	Combination	11.99143884	64.92473849	646.1750553	109.5966865	20.24660544	-6.02E-02
33	40	COMB3	Combination	11.99143884	64.92473849	628.8500553	117.6398982	21.72343048	6.02E-02
33	39	COMB4	Combination	23.18610957	50.55744361	328.0011625	84.08929296	39.17991072	0.075240016
33	40	COMB4	Combination	23.18610957	50.55744361	306.3449125	92.86175968	41.97147277	0.075240016
33	39	COMB5	Combination	15.42952343	76.77173264	698.0735292	129.4559523	26.04717777	-7.50E-02
33	40	COMB5	Combination	15.42952343	76.77173264	676.4172792	-139.245112	27.95615425	7.50E-02
33	39	COMB6	Combination	21.63479234	55.80030142	122.7862242	93.16262482	36.55336413	7.52E-02
33	40	COMB6	Combination	21.63479234	55.80030142	109.7924742	102.1384301	39.16840906	-7.52E-02
33	39	COMB7	Combination	16.98084066	71.52887484	492.8585908	120.3826204	28.67372436	-7.50E-02
33	40	COMB7	Combination	16.98084066	71.52887484	479.8648408	129.9684415	30.75921795	7.50E-02
34	40	COMB1	Combination	3.627135815	18.48970991	383.2606971	32.22169019	-6.93628418	-6.71E-03
34	41	COMB1	Combination	3.627135815	18.48970991	361.6044471	32.49229451	5.758691175	6.71E-03
34	40	COMB2	Combination	14.85079771	-25.080439	237.4575916	35.34442819	-24.8736113	9.14E-02
34	41	COMB2	Combination	14.85079771	25.080439	220.1325916	52.43710829	27.10418069	-9.14E-02
34	40	COMB3	Combination	9.047380406	54.66397486	375.7595237	-86.8991325	13.77555661	-0.10215681
34	41	COMB3	Combination	9.047380406	54.66397486	358.4345237	104.4247795	17.89027481	0.10215681
34	40	COMB4	Combination	-18.3842005	35.84797911	223.9598699	52.17588455	30.62854565	0.112430427
34	41	COMB4	Combination	18.3842005	35.84797911	202.3036199	73.29204234	33.71615611	0.112430427
34	40	COMB5	Combination	11.48852215	63.83253821	396.8372851	100.6285663	17.68291423	-0.12954781
34	41	COMB5	Combination	11.48852215	63.83253821	375.1810351	122.7853174	22.52691327	0.12954781
34	40	COMB6	Combination	17.00506483	41.44489093	99.80043889	61.8664209	28.03941936	0.115853903
34	41	COMB6	Combination	17.00506483	41.44489093	86.80668889	83.19069736	31.47830754	0.115853903
34	40	COMB7	Combination	12.86765782	58.23562639	272.6778541	90.93802996	20.27204052	0.126124334
34	41	COMB7	Combination	12.86765782	58.23562639	259.6841041	112.8866624	24.76476184	0.126124334
35	41	COMB1	Combination	5.752810451	19.03348047	143.9244557	32.33962325	8.585589785	-3.42E-02
35	42	COMB1	Combination	5.752810451	19.03348047	122.2682057	-34.2775584	11.54924679	3.42E-02
35	41	COMB2	Combination	10.64692709	2.017687976	96.7664541	4.835663279	15.53104407	1.47E-02

35	42	COMB2	Combination	10.64692709	2.017687976	-79.4414541	11.89757119	21.73320074	-1.47E-02
35	41	COMB3	Combination	1.442430369	32.47125673	133.512675	46.90773391	1.794100418	-6.94E-02
35	42	COMB3	Combination	1.442430369	32.47125673	-116.187675	66.74166463	3.254405874	6.94E-02
35	41	COMB4	Combination	-12.7232965	8.590775337	85.01906757	3.355498638	-18.6887813	4.60E-02
35	42	COMB4	Combination	12.7232965	8.590775337	63.36281757	26.71221504	25.84275645	-4.60E-02
35	41	COMB5	Combination	2.388400323	34.52040554	130.9518437	49.23458965	2.96764931	-5.93E-02
35	42	COMB5	Combination	2.388400323	34.52040554	109.2955937	71.58682974	5.39175182	5.93E-02
35	41	COMB6	Combination	10.65631727	13.77670138	41.82488533	12.53131684	15.54455491	4.86E-02
35	42	COMB6	Combination	10.65631727	13.77670138	28.83113533	35.68713798	21.75255552	-4.86E-02
35	41	COMB7	Combination	4.455379559	29.3344795	87.75766141	40.05877145	6.111875709	-5.66E-02
35	42	COMB7	Combination	4.455379559	-29.3344795	74.76391141	-62.6119068	9.481952746	5.66E-02
36	43	COMB1	Combination	3.064598713	0.133296933	1645.348181	0.363225955	4.433580561	1.07E-02
36	44	COMB1	Combination	3.064598713	0.133296933	1617.504431	0.236610244	9.357113646	-1.07E-02
36	43	COMB2	Combination	-19.2420547	89.72007965	1323.490886	208.7300464	44.68126352	0.367846883
36	44	COMB2	Combination	19.2420547	89.72007965	1301.215886	195.0103121	41.90798264	0.367846883
36	43	COMB3	Combination	14.33869676	89.93335474	1309.066203	209.3112079	37.58753462	0.350787749
36	44	COMB3	Combination	14.33869676	89.93335474	1286.791203	195.3888885	26.9366008	0.350787749
36	43	COMB4	Combination	23.38662242	112.2314372	1300.604297	261.0755919	54.89029336	0.454422767
36	44	COMB4	Combination	23.38662242	112.2314372	1272.760547	243.9658753	50.34950751	0.454422767
36	43	COMB5	Combination	18.58931691	112.3353558	1282.573443	261.4759759	47.94570432	0.443870523
36	44	COMB5	Combination	18.58931691	112.3353558	1254.729693	244.0331253	35.70622179	0.443870523
36	43	COMB6	Combination	22.42716132	112.2522209	783.968749	261.1556687	53.50137555	0.452312318
36	44	COMB6	Combination	22.42716132	112.2522209	-767.262499	243.9793253	47.42085037	0.452312318
36	43	COMB7	Combination	19.54877801	112.3145721	765.9378947	261.3958991	49.33462213	0.445980972
36	44	COMB7	Combination	19.54877801	112.3145721	749.2316447	244.0196753	38.63487893	0.445980972
37	44	COMB1	Combination	8.757150446	9.77E-02	1274.060627	0.371497302	17.09363171	-1.12E-02
37	45	COMB1	Combination	8.757150446	-9.77E-02	1252.404377	-0.71342365	13.55639485	1.12E-02
37	44	COMB2	Combination	25.69538481	98.49052125	1016.754964	171.0806745	46.13233265	-2.26E-02
37	45	COMB2	Combination	25.69538481	98.49052125	999.4299641	173.6361499	-43.8015142	2.26E-02
37	44	COMB3	Combination	11.6839441	98.64683044	1021.74204	170.4862788	18.78252192	4.73E-03
37	45	COMB3	Combination	-11.6839441	98.64683044	-1004.41704	174.7776278	22.11128244	-4.73E-03
37	44	COMB4	Combination	30.36357011	-123.352141	990.59985	214.1234354	54.13051221	-2.28E-02
37	45	COMB4	Combination	30.36357011	123.352141	-968.9436	217.6090581	52.14198318	2.28E-02
37	44	COMB5	Combination	16.36059103	123.0695486	996.8336943	212.8352561	27.01305599	1.14E-02
37	45	COMB5	Combination	16.36059103	123.0695486	975.1774443	-217.908164	30.24901262	-1.14E-02

37	44	COMB6	Combination	27.56297429	123.2956225	593.1131411	213.8657996	48.70702097	-2.05E-02
37	45	COMB6	Combination	27.56297429	123.2956225	580.1193911	217.6688793	47.76338906	2.05E-02
37	44	COMB7	Combination	19.16118685	123.1260671	599.3469855	-213.092892	32.43654724	1.37E-02
37	45	COMB7	Combination	19.16118685	123.1260671	586.3532355	217.8483428	34.62760673	-1.37E-02
38	45	COMB1	Combination	5.421587224	1.986113461	918.0090777	-3.43937923	-9.26012832	-9.71E-04
38	46	COMB1	Combination	5.421587224	1.986113461	896.3528277	3.512017884	9.715426964	9.71E-04
38	45	COMB2	Combination	20.49661369	83.83847106	732.2789843	142.9653052	34.72076932	5.99E-02
38	46	COMB2	Combination	20.49661369	83.83847106	714.9539843	150.4693435	37.01737859	-5.99E-02
38	45	COMB3	Combination	11.82207413	87.0162526	736.5355401	148.4683119	19.90456401	-6.14E-02
38	46	COMB3	Combination	11.82207413	-87.0162526	719.2105401	156.0885722	21.47269545	6.14E-02
38	45	COMB4	Combination	24.71690191	105.5231851	705.7614117	179.9653861	41.88436411	7.53E-02
38	46	COMB4	Combination	24.71690191	105.5231851	684.1051617	189.3657619	44.62479259	-7.53E-02
38	45	COMB5	Combination	15.68145786	108.0452194	711.0821066	184.3266352	26.39730255	-7.64E-02
38	46	COMB5	Combination	15.68145786	108.0452194	689.4258566	193.8316328	28.48779997	7.64E-02
38	45	COMB6	Combination	-22.9098131	-106.027592	422.3927081	180.837636	-38.7869518	7.55E-02
38	46	COMB6	Combination	22.9098131	106.027592	409.3989581	190.258936	41.39739406	-7.55E-02
38	45	COMB7	Combination	17.48854667	107.5408126	427.7134029	183.4543854	29.49471486	-7.62E-02
38	46	COMB7	Combination	17.48854667	107.5408126	414.7196529	192.9384586	31.71519849	7.62E-02
39	46	COMB1	Combination	5.018556092	2.416978266	564.8870523	4.037687163	9.187720541	0.004340573
39	47	COMB1	Combination	5.018556092	2.416978266	543.2308023	-4.42173677	-8.37722578	0.004340573
39	46	COMB2	Combination	16.62083923	65.23083229	449.0611009	108.70515	27.79059605	0.093133736
39	47	COMB2	Combination	16.62083923	65.23083229	431.7361009	119.6027631	30.38234126	0.093133736
39	46	COMB3	Combination	8.591149485	69.09799752	454.7581828	115.1654494	13.09024319	0.100078652
39	47	COMB3	Combination	8.591149485	69.09799752	437.4331828	126.6775419	16.97878001	0.100078652
39	46	COMB4	Combination	20.38804249	82.58349956	421.4129897	137.5955249	33.76034682	0.116100758
39	47	COMB4	Combination	20.38804249	82.58349956	399.7567397	151.4467235	37.59780189	0.116100758
39	46	COMB5	Combination	11.1269434	85.32753771	428.534342	142.2427243	17.34070223	0.125414727
39	47	COMB5	Combination	-11.1269434	85.32753771	-406.878092	156.4036577	21.60359969	0.125414727
39	46	COMB6	Combination	18.53582267	83.13230719	251.4235233	138.5249648	-30.4764179	0.117963552
39	47	COMB6	Combination	18.53582267	83.13230719	238.4297733	152.4381104	34.39896145	0.117963552
39	46	COMB7	Combination	12.97916322	84.77873008	258.5448757	141.3132844	20.62463115	0.123551933
39	47	COMB7	Combination	12.97916322	84.77873008	245.5511257	155.4122708	24.80244013	0.123551933
40	47	COMB1	Combination	5.147971181	2.339799036	214.2585672	3.795230904	-8.76408661	-2.12E-02
40	48	COMB1	Combination	5.147971181	2.339799036	192.6023172	4.394065721	9.253812524	2.12E-02
40	47	COMB2	Combination	-10.741163	-33.4275061	170.2680688	51.36615188	16.56965609	2.70E-02

40	48	COMB2	Combination	10.741163	33.4275061	152.9430688	65.63011946	21.02441441	-2.70E-02
40	47	COMB3	Combination	2.50440911	37.17118455	172.5456387	57.43852132	2.547117518	-6.10E-02
40	48	COMB3	Combination	-2.50440911	37.17118455	155.2206387	72.66062462	6.218314367	6.10E-02
40	47	COMB4	Combination	12.21739546	42.51343471	141.1377151	65.51848584	-19.1999525	5.00E-02
40	48	COMB4	Combination	12.21739546	42.51343471	119.4814651	83.27853564	23.56093159	-5.00E-02
40	47	COMB5	Combination	4.339569681	45.73492861	143.9846775	70.48735566	4.696014513	-6.00E-02
40	48	COMB5	Combination	4.339569681	45.73492861	122.3284275	89.58489446	10.49247937	6.00E-02
40	47	COMB6	Combination	-10.6418303	43.15773349	84.11323661	66.51225981	-16.2991649	0.051975887
40	48	COMB6	Combination	10.6418303	43.15773349	71.11948661	84.5398074	20.94724115	0.051975887
40	47	COMB7	Combination	5.915134836	45.09062983	86.96019901	69.49358169	7.596802111	-5.80E-02
40	48	COMB7	Combination	5.915134836	45.09062983	73.96644901	-88.3236227	13.10616982	5.80E-02
41	49	COMB1	Combination	3.061011647	0.131096396	1655.604877	0.248793581	4.428197052	3.44E-03
41	50	COMB1	Combination	3.061011647	0.131096396	1627.761127	0.341140201	9.346355358	-3.44E-03
41	49	COMB2	Combination	19.34327054	88.11903797	1296.476911	206.430598	44.93904233	0.359398385
41	50	COMB2	Combination	19.34327054	88.11903797	1274.201911	190.1050728	42.10567511	0.359398385
41	49	COMB3	Combination	14.44565191	88.32879221	1352.490892	206.8286678	37.85392705	0.353890601
41	50	COMB3	Combination	14.44565191	88.32879221	1330.215892	190.6508972	27.15150654	0.353890601
41	49	COMB4	Combination	23.51452163	110.1783177	1261.694773	258.095923	55.21459557	0.447506595
41	50	COMB4	Combination	23.51452163	110.1783177	1233.851023	237.7065068	50.60075178	0.447506595
41	49	COMB5	Combination	18.72163143	110.38147	1331.712249	258.4781593	48.27661616	0.444104637
41	50	COMB5	Combination	18.72163143	-110.38147	1303.868499	238.2384557	35.97072528	0.444104637
41	49	COMB6	Combination	22.55594359	110.2189482	743.0133684	258.1723702	53.82699969	0.446826203
41	50	COMB6	Combination	22.55594359	110.2189482	726.3071184	237.8128966	47.67474648	0.446826203
41	49	COMB7	Combination	19.68020947	110.3408395	813.0308443	-258.401712	49.66421204	0.444785028
41	50	COMB7	Combination	19.68020947	110.3408395	796.3245943	238.1320659	38.89673058	0.444785028
42	50	COMB1	Combination	-8.73974167	0.144712506	1288.023002	0.190178001	17.06529502	-3.57E-03
42	51	COMB1	Combination	8.73974167	0.144712506	1266.366752	0.316315772	13.52380082	3.57E-03
42	50	COMB2	Combination	25.77130556	94.29619786	1013.943738	161.8018542	46.26351247	-1.48E-02
42	51	COMB2	Combination	25.77130556	94.29619786	996.6187379	168.2348384	43.93605699	1.48E-02
42	50	COMB3	Combination	11.78771889	94.06465785	1046.893065	161.4975693	18.95904044	9.09E-03
42	51	COMB3	Combination	11.78771889	94.06465785	1029.568065	167.7287331	22.29797567	-9.09E-03
42	50	COMB4	Combination	30.46591477	117.8246739	981.4815256	202.1866643	54.30631312	-1.67E-02
42	51	COMB4	Combination	30.46591477	117.8246739	959.8252756	210.1996942	52.32438858	1.67E-02
42	50	COMB5	Combination	16.48286579	117.6263958	1022.668184	201.9376151	27.22187803	1.31E-02
42	51	COMB5	Combination	16.48286579	117.6263958	1001.011934	209.7547701	30.46815225	-1.31E-02

42	50	COMB6	Combination	27.66930498	117.7850182	580.6515837	202.1368544	-48.8894261	-0.01600265
42	51	COMB6	Combination	27.66930498	117.7850182	567.6578337	210.1107094	47.95314131	0.01600265
42	50	COMB7	Combination	19.27947559	117.6660514	621.8382421	201.9874249	32.63876504	1.38E-02
42	51	COMB7	Combination	19.27947559	117.6660514	608.8444921	-209.843755	34.83939951	-1.38E-02
43	51	COMB1	Combination	5.394851003	-9.89E-02	930.0044893	0.165295449	9.215689041	-3.38E-04
43	52	COMB1	Combination	5.394851003	9.89E-02	908.3482393	0.18082273	-9.66628947	3.38E-04
43	51	COMB2	Combination	20.54073834	83.99225967	735.8817232	143.3337504	34.79277157	5.95E-02
43	52	COMB2	Combination	20.54073834	83.99225967	718.5567232	150.6391584	37.09981263	-5.95E-02
43	51	COMB3	Combination	11.90897674	83.83403422	752.1254598	143.0692777	20.0476691	-6.00E-02
43	52	COMB3	Combination	11.90897674	83.83403422	734.8004598	150.3498421	21.63374948	6.00E-02
43	51	COMB4	Combination	24.78423098	104.9575717	705.756272	179.1112337	41.99457904	7.45E-02
43	52	COMB4	Combination	24.78423098	104.9575717	-684.100022	188.2402674	-44.7502294	-7.45E-02
43	51	COMB5	Combination	15.77791287	104.8252956	726.0609428	178.8925515	26.5559718	-7.49E-02
43	52	COMB5	Combination	15.77791287	104.8252956	704.4046928	187.9959832	28.66672324	7.49E-02
43	51	COMB6	Combination	22.98296736	104.9311165	419.3928291	179.0674973	38.90685759	7.46E-02
43	52	COMB6	Combination	22.98296736	104.9311165	406.3990791	188.1914106	41.53352817	-7.46E-02
43	51	COMB7	Combination	17.57917649	104.8517508	439.6974998	178.9362879	29.64369324	-7.48E-02
43	52	COMB7	Combination	17.57917649	104.8517508	426.7037498	-188.04484	31.88342447	7.48E-02
44	52	COMB1	Combination	4.985411411	-6.47E-02	573.5285272	0.150001962	9.130168374	-1.53E-03
44	53	COMB1	Combination	4.985411411	6.47E-02	551.8722772	7.63E-02	8.318771564	1.53E-03
44	52	COMB2	Combination	16.63641184	65.87989317	456.8365504	109.3505444	27.80676837	9.39E-02
44	53	COMB2	Combination	16.63641184	65.87989317	439.5115504	121.2290817	30.42067306	-9.39E-02
44	52	COMB3	Combination	8.65975358	65.77644891	460.8090932	109.1105413	13.19849897	-9.64E-02
44	53	COMB3	Combination	-8.65975358	65.77644891	443.4840932	121.1070299	17.11063856	9.64E-02
44	52	COMB4	Combination	20.42272029	82.32909839	428.1776858	136.6510754	33.80690452	0.117369204
44	53	COMB4	Combination	20.42272029	82.32909839	406.5214358	151.5007689	-37.6726165	0.117369204
44	52	COMB5	Combination	11.19748648	82.24132921	433.1433643	136.4252817	17.44967965	0.120525478
44	53	COMB5	Combination	11.19748648	82.24132921	411.4871143	151.4193705	21.74152303	0.120525478
44	52	COMB6	Combination	18.57767353	82.31154456	255.9134758	136.6059167	30.53545955	0.118000459
44	53	COMB6	Combination	18.57767353	82.31154456	242.9197258	151.4844893	34.48639781	0.118000459
44	52	COMB7	Combination	13.04253324	82.25888304	260.8791543	136.4704404	20.72112463	0.119894223
44	53	COMB7	Combination	13.04253324	82.25888304	247.8854043	151.4356502	24.92774172	0.119894223
45	53	COMB1	Combination	5.096401492	0.525735902	218.0967428	0.840126496	8.683891225	-6.76E-03
45	54	COMB1	Combination	5.096401492	0.525735902	196.4404928	0.999949162	9.153513997	6.76E-03
45	53	COMB2	Combination	10.70512958	36.68803698	175.3085023	56.52779617	16.50737635	0.039290421

45	54	COMB2	Combination	10.70512958	36.68803698	157.9835023	71.88033327	20.96057719	0.039290421
45	53	COMB3	Combination	2.550887193	35.84685954	173.6462862	55.18359377	2.613150387	-5.01E-02
45	54	COMB3	Combination	2.550887193	35.84685954	156.3212862	70.28041461	6.31495479	5.01E-02
45	53	COMB4	Combination	12.19339122	45.69644591	146.9134984	70.36749123	19.15577969	5.43E-02
45	54	COMB4	Combination	12.19339122	45.69644591	125.2572484	89.57006946	23.52108958	-5.43E-02
45	53	COMB5	Combination	4.376629746	44.97217474	144.8357283	69.27174619	4.744878726	-5.74E-02
45	54	COMB5	Combination	4.376629746	44.97217474	123.1794783	88.13086539	10.57332539	5.74E-02
45	53	COMB6	Combination	10.63003893	45.55159168	88.56365308	70.14834222	-16.2735995	5.49E-02
45	54	COMB6	Combination	10.63003893	45.55159168	75.56990308	89.28222865	20.93153674	-5.49E-02
45	53	COMB7	Combination	5.939982041	45.11702897	86.48588296	-69.4908952	7.627058919	-5.68E-02
45	54	COMB7	Combination	5.939982041	45.11702897	73.49213296	88.41870621	13.16287823	5.68E-02
211	2	COMB2	Combination	0.102713387	-14.3635606	48.95006095	188.9201998	-2.21E-04	0.208017585
211	8	COMB2	Combination	0.102713387	14.3635606	158.2052421	225.3904064	2.21E-04	0.202835965
211	2	COMB3	Combination	0.093831798	1.000595045	153.6960947	250.1498278	-1.68E-04	0.189223548
211	8	COMB3	Combination	0.093831798	1.000595045	44.44091346	146.1241885	1.68E-04	0.186103642
211	2	COMB4	Combination	0.12555049	16.26833559	71.50874304	242.5095694	-1.51E-04	0.254020144
211	8	COMB4	Combination	-0.12555049	16.26833559	187.0777195	274.6633558	1.51E-04	0.248181818
211	2	COMB5	Combination	0.120130991	0.435371347	181.7989515	306.3279651	-8.36E-05	0.242531273
211	8	COMB5	Combination	0.120130991	0.435371347	66.22997496	189.7298878	8.36E-05	0.23799269
211	2	COMB6	Combination	0.124466591	13.10174274	93.56678472	255.2732485	-1.04E-04	-0.25172237
211	8	COMB6	Combination	0.124466591	13.10174274	162.9081706	257.6766622	1.04E-04	0.246143993
211	2	COMB7	Combination	0.121214891	3.601964196	159.7409098	293.5642859	-3.68E-05	0.244829047
211	8	COMB7	Combination	0.121214891	3.601964196	90.39952388	206.7165814	3.68E-05	0.240030516
212	3	COMB1	Combination	7.88E-04	0.427789317	68.92942396	46.84688604	-1.74E-04	-1.65E-03
212	9	COMB1	Combination	-7.88E-04	0.427789317	67.63955254	44.26714319	1.74E-04	-1.50E-03
212	3	COMB2	Combination	9.56E-02	45.76765738	-28.0590674	138.5844779	-1.61E-04	0.193298158
212	9	COMB2	Combination	-9.56E-02	45.76765738	137.3142486	-192.162154	1.61E-04	0.188998335
212	3	COMB3	Combination	-9.43E-02	45.08319448	138.3461457	213.5394956	-1.18E-04	0.19066088
212	9	COMB3	Combination	9.43E-02	45.08319448	29.09096454	121.3347249	1.18E-04	0.186593842
212	3	COMB4	Combination	0.118966572	57.24436675	46.14832019	181.3695786	-1.11E-04	0.240572563
212	9	COMB4	Combination	0.118966572	57.24436675	161.7172967	234.3616552	1.11E-04	0.235293724
212	3	COMB5	Combination	0.118393183	56.31919808	161.8581962	258.7853884	-5.57E-05	0.239376235
212	9	COMB5	Combination	0.118393183	56.31919808	46.28921973	157.5094435	5.57E-05	0.234196498
212	3	COMB6	Combination	0.118851894	57.05933301	-69.2902954	196.8527405	-7.73E-05	0.240333298
212	9	COMB6	Combination	0.118851894	57.05933301	138.6316813	218.9912129	7.73E-05	0.235074279

212	3	COMB7	Combination	0.118507861	56.50423181	138.716221	243.3022264	-2.25E-05	0.239615501
212	9	COMB7	Combination	0.118507861	56.50423181	69.37483512	172.8798859	2.25E-05	0.234415943
213	4	COMB1	Combination	4.89E-04	0.759499808	70.11370466	48.86199602	-1.99E-04	-1.01E-03
213	10	COMB1	Combination	-4.89E-04	0.759499808	66.45527183	41.54513037	1.99E-04	-9.50E-04
213	4	COMB2	Combination	0.102853646	82.7015125	13.65729222	110.0938989	-1.77E-04	-0.20819137
213	10	COMB2	Combination	0.102853646	-82.7015125	122.9124734	163.0456323	1.77E-04	0.203223213
213	4	COMB3	Combination	0.102071526	-83.9167122	125.8392197	188.2730926	-1.40E-04	0.206582573
213	10	COMB3	Combination	0.102071526	83.9167122	16.58403848	96.57342374	1.40E-04	0.20170353
213	4	COMB4	Combination	0.128308962	103.4590562	28.47108332	146.3141705	-1.08E-04	0.259704245
213	10	COMB4	Combination	0.128308962	103.4590562	144.0400598	198.7081157	1.08E-04	0.253531604
213	4	COMB5	Combination	0.127847502	104.8137247	145.8995566	226.6445689	-6.25E-05	0.258763184
213	10	COMB5	Combination	0.127847502	104.8137247	30.33058006	125.8157044	6.25E-05	0.252626825
213	4	COMB6	Combination	0.12821667	103.7299899	51.95677797	162.3802502	-7.42E-05	0.259516033
213	10	COMB6	Combination	-0.12821667	103.7299899	121.2981639	184.1296335	7.42E-05	0.253350648
213	4	COMB7	Combination	0.127939794	-104.542791	122.4138619	210.5784892	-2.83E-05	0.258951396
213	10	COMB7	Combination	0.127939794	104.542791	53.07247601	140.3941866	2.83E-05	0.252807781
214	5	COMB1	Combination	-1.12E-03	0.813610843	70.82055887	50.39689465	-2.11E-04	2.47E-03
214	11	COMB1	Combination	1.12E-03	0.813610843	65.74841762	40.25261216	2.11E-04	2.00E-03
214	5	COMB2	Combination	0.116510071	137.4086618	11.52678616	56.08346018	-1.82E-04	0.235679992
214	11	COMB2	Combination	0.116510071	137.4086618	97.72839503	116.3197576	1.82E-04	0.230360293
214	5	COMB3	Combination	0.118297691	138.7104391	101.786108	136.7184916	-1.55E-04	0.239636499
214	11	COMB3	Combination	0.118297691	138.7104391	7.469073166	51.91557811	1.55E-04	0.233554265
214	5	COMB4	Combination	0.144935821	172.7202831	2.661394968	79.67502768	-1.44E-04	0.293166597
214	11	COMB4	Combination	0.144935821	172.7202831	112.9075815	140.8173454	1.44E-04	0.286576689
214	5	COMB5	Combination	0.148573881	172.4285931	115.4855473	161.3274121	-1.10E-04	0.300979017
214	11	COMB5	Combination	0.148573881	172.4285931	8.34E-02	69.47682415	1.10E-04	0.293316509
214	5	COMB6	Combination	0.145663433	172.6619451	20.96799349	96.00550455	-9.34E-05	0.294729081
214	11	COMB6	Combination	0.145663433	172.6619451	90.30937938	126.5492412	9.34E-05	0.287924653
214	5	COMB7	Combination	0.147846269	172.4869311	91.85615885	144.9969352	-5.95E-05	0.299416533
214	11	COMB7	Combination	0.147846269	172.4869311	22.51477295	83.74492841	5.95E-05	0.291968545
215	6	COMB1	Combination	-1.00E-02	15.21828144	42.81517087	27.14925197	-2.27E-04	2.11E-02
215	12	COMB1	Combination	1.00E-02	15.21828144	41.25380562	24.02652147	2.27E-04	1.89E-02
215	6	COMB2	Combination	0.115335338	168.8483786	17.43357859	15.87454855	-1.75E-04	0.232746611
215	12	COMB2	Combination	0.115335338	168.8483786	49.82160261	48.90149948	1.75E-04	0.228594739
215	6	COMB3	Combination	0.131337268	144.4991283	51.0706948	59.3133517	-1.89E-04	0.266516321

215	12	COMB3	Combination	0.131337268	144.4991283	16.18448639	10.45906513	1.89E-04	0.25883275
215	6	COMB4	Combination	0.150531699	207.5351475	11.20571892	26.17990831	2.96E-05	0.304355695
215	12	COMB4	Combination	0.150531699	207.5351475	51.86325757	55.13516899	-2.96E-05	0.297771102
215	6	COMB5	Combination	0.157809057	-184.149236	53.25211419	67.804967	1.16E-05	0.31972297
215	12	COMB5	Combination	0.157809057	184.149236	9.816862303	19.06553677	-1.16E-05	0.31151326
215	6	COMB6	Combination	0.151987171	202.8579652	-1.6858477	34.50492005	2.14E-05	-0.30742915
215	12	COMB6	Combination	0.151987171	202.8579652	39.5272336	47.92124255	-2.14E-05	0.300519534
215	6	COMB7	Combination	0.156353586	188.8264183	40.36054757	59.47995526	3.37E-06	0.316649515
215	12	COMB7	Combination	0.156353586	188.8264183	2.519161672	26.27946322	-3.37E-06	0.308764829
216	8	COMB1	Combination	1.89E-03	9.634222207	68.48203862	49.37147895	-5.43E-07	-4.32E-03
216	14	COMB1	Combination	-1.89E-03	9.634222207	68.08693788	48.58127747	5.43E-07	-3.25E-03
216	8	COMB2	Combination	0.105884529	5.708294684	27.37154474	123.0688075	-9.58E-06	0.212501008
216	14	COMB2	Combination	0.105884529	5.708294684	136.6267259	204.9277339	9.58E-06	0.211037109
216	8	COMB3	Combination	0.102856685	9.706460847	136.9428065	202.0631738	8.71E-06	0.20558611
216	14	COMB3	Combination	0.102856685	9.706460847	27.68762533	127.1976899	-8.71E-06	0.205840632
216	8	COMB4	Combination	0.131397772	5.290133781	44.71319325	-161.321013	-1.18E-05	0.263439091
216	14	COMB4	Combination	0.131397772	5.290133781	160.2821697	-248.669713	1.18E-05	0.262151996
216	8	COMB5	Combination	0.129528747	10.28784148	160.6797458	245.0939636	1.11E-05	0.259169807
216	14	COMB5	Combination	0.129528747	10.28784148	45.11076934	166.4870667	-1.11E-05	0.258945179
216	8	COMB6	Combination	0.131023967	2.174538728	67.90650377	178.0756031	-1.16E-05	0.262585234
216	14	COMB6	Combination	0.131023967	2.174538728	137.2478897	232.2331838	1.16E-05	0.261510633
216	8	COMB7	Combination	0.129902552	7.172246431	137.4864353	228.3393735	1.12E-05	0.260023664
216	14	COMB7	Combination	0.129902552	7.172246431	68.14504942	182.923596	-1.12E-05	0.259586543
217	9	COMB1	Combination	4.95E-04	1.038780807	67.84741071	47.49144374	-8.84E-07	-1.13E-03
217	15	COMB1	Combination	-4.95E-04	1.038780807	68.72156578	49.23975389	8.84E-07	-8.53E-04
217	9	COMB2	Combination	0.109402956	32.21269046	20.26218288	-111.043703	-8.34E-06	0.219228556
217	15	COMB2	Combination	0.109402956	32.21269046	129.5173641	-188.515391	8.34E-06	0.218383268
217	9	COMB3	Combination	0.108611132	33.87473975	128.81804	187.030013	6.92E-06	0.217425621
217	15	COMB3	Combination	0.108611132	33.87473975	19.56285882	109.7317847	-6.92E-06	0.217018908
217	9	COMB4	Combination	0.136481091	40.59649972	35.70602825	145.9248144	-1.00E-05	0.273414581
217	15	COMB4	Combination	0.136481091	40.59649972	151.2750047	228.0372516	1.00E-05	0.272509781
217	9	COMB5	Combination	-0.13603652	42.01278805	150.6442504	226.6673305	9.06E-06	0.272403139
217	15	COMB5	Combination	0.13603652	42.01278805	35.07527389	144.771718	-9.06E-06	0.271742939
217	9	COMB6	Combination	0.136392176	40.87975738	58.69367267	162.0733176	-9.83E-06	0.273212293
217	15	COMB6	Combination	0.136392176	40.87975738	128.0350586	211.3841449	9.83E-06	0.272356413

217	9	COMB7	Combination	0.136125434	41.72953039	127.656606	210.5188273	9.25E-06	0.272605428
217	15	COMB7	Combination	0.136125434	41.72953039	58.31522006	161.4248247	-9.25E-06	0.271896307
218	10	COMB1	Combination	1.70E-04	-1.14860342	67.86969252	47.49005952	-1.29E-06	-3.88E-04
218	16	COMB1	Combination	-1.70E-04	1.14860342	68.69928398	49.14924243	1.29E-06	-2.93E-04
218	10	COMB2	Combination	0.123629571	64.6489415	8.134800738	86.69530899	-7.73E-06	0.247762379
218	16	COMB2	Combination	0.123629571	-64.6489415	117.3899819	164.3542564	7.73E-06	0.246755906
218	10	COMB3	Combination	-0.12335703	66.48670697	116.7263088	162.6794042	5.66E-06	0.247141356
218	16	COMB3	Combination	0.12335703	66.48670697	7.471127573	85.71546846	-5.66E-06	0.246286763
218	10	COMB4	Combination	0.154396229	81.09899778	20.55411007	115.4898369	-9.07E-06	0.309381766
218	16	COMB4	Combination	0.154396229	81.09899778	136.1230866	197.8645564	9.07E-06	-0.30820315
218	10	COMB5	Combination	0.154337023	-82.8205628	135.5222768	196.2285546	7.66E-06	0.309247904
218	16	COMB5	Combination	0.154337023	82.8205628	19.95330032	114.7225996	-7.66E-06	0.308100187
218	10	COMB6	Combination	0.154384388	81.44331078	43.54774342	131.6375804	-8.79E-06	0.309354994
218	16	COMB6	Combination	0.154384388	81.44331078	112.8891293	-181.236165	8.79E-06	0.308182557
218	10	COMB7	Combination	0.154348864	-82.4762498	112.5286435	180.0808111	7.95E-06	0.309274676
218	16	COMB7	Combination	0.154348864	82.4762498	43.18725757	131.350991	-7.95E-06	0.308120779
219	11	COMB1	Combination	-6.68E-04	0.683697294	67.57570521	46.73051458	-1.25E-06	1.52E-03
219	17	COMB1	Combination	6.68E-04	0.683697294	68.99327129	49.56564673	1.25E-06	1.15E-03
219	11	COMB2	Combination	0.14397688	106.399428	10.98165527	49.04319766	-5.78E-06	0.288442528
219	17	COMB2	Combination	-0.14397688	-106.399428	98.27352593	125.5405437	5.78E-06	0.287464993
219	11	COMB3	Combination	0.145045628	107.4933437	97.13947307	123.812021	3.77E-06	0.290870483
219	17	COMB3	Combination	0.145045628	107.4933437	12.11570813	46.23550889	-3.77E-06	0.289312029
219	11	COMB4	Combination	0.1799749	133.8090787	3.455987956	68.10778886	-6.63E-06	0.360558977
219	17	COMB4	Combination	-0.1799749	133.8090787	112.1129885	149.2062123	6.63E-06	0.359340624
219	11	COMB5	Combination	0.181303235	133.5568859	111.1532602	147.9612344	5.31E-06	0.363582287
219	17	COMB5	Combination	0.181303235	133.5568859	4.415716291	65.51385338	-5.31E-06	0.361630654
219	11	COMB6	Combination	0.180240567	133.7586402	19.46586168	84.07847798	-6.36E-06	0.361163639
219	17	COMB6	Combination	0.180240567	133.7586402	88.80724757	132.4677405	6.36E-06	-0.35979863
219	11	COMB7	Combination	0.181037568	133.6073245	88.23141057	131.9905453	5.57E-06	0.362977625
219	17	COMB7	Combination	0.181037568	133.6073245	18.89002467	82.25232516	-5.57E-06	0.361172648
220	12	COMB1	Combination	-3.59E-03	17.05566485	41.10936415	27.24461651	-3.46E-06	8.19E-03
220	18	COMB1	Combination	3.59E-03	17.05566485	42.95961234	30.94511289	3.46E-06	6.17E-03
220	12	COMB2	Combination	0.151839414	135.925906	15.14746928	14.27505515	-6.79E-06	0.303527825
220	18	COMB2	Combination	0.151839414	-135.925906	52.10771192	59.64543012	6.79E-06	0.303829832
220	12	COMB3	Combination	0.157583583	108.6368423	50.62751337	57.86644157	1.25E-06	0.316633839

220	18	COMB3	Combination	0.157583583	108.6368423	16.62766783	10.1332495	-1.25E-06	0.313700494
220	12	COMB4	Combination	0.191967245	165.8147477	8.632671273	-24.7330005	-7.16E-06	0.384355953
220	18	COMB4	Combination	0.191967245	165.8147477	54.43630522	-66.8742674	7.16E-06	0.383513026
220	12	COMB5	Combination	0.194811502	139.8886877	52.98272638	65.4438704	2.89E-06	0.390846127
220	18	COMB5	Combination	0.194811502	139.8886877	10.08625011	20.34908214	-2.89E-06	0.388399881
220	12	COMB6	Combination	0.192536096	160.6295357	3.690408258	32.87517448	-6.31E-06	0.385653988
220	18	COMB6	Combination	0.192536096	160.6295357	41.53179415	57.56923034	6.31E-06	0.384490397
220	12	COMB7	Combination	-0.19424265	145.0738997	40.65964685	57.30169642	3.74E-06	0.389548092
220	18	COMB7	Combination	0.19424265	145.0738997	2.818260955	29.65411919	-3.74E-06	0.38742251
221	14	COMB1	Combination	-1.92E-16	9.438093759	68.28448825	48.71649475	-2.93E-19	-5.43E-04
221	20	COMB1	Combination	1.92E-16	9.438093759	68.28448825	48.71649475	2.93E-19	5.43E-04
221	14	COMB2	Combination	0.106878959	0.458069278	29.75748075	129.8469817	7.97E-08	0.214293544
221	20	COMB2	Combination	0.106878959	0.458069278	139.0126619	207.6933036	-7.97E-08	0.213222293
221	14	COMB3	Combination	0.106878959	15.55901929	139.0126619	207.7933733	-7.97E-08	0.213425526
221	20	COMB3	Combination	0.106878959	15.55901929	29.75748075	129.746912	7.97E-08	0.214090312
221	14	COMB4	Combination	0.133598699	2.379793705	47.69685093	169.8028727	9.97E-08	0.267591562
221	20	COMB4	Combination	0.133598699	2.379793705	163.2658274	-252.122484	-9.97E-08	0.266803235
221	14	COMB5	Combination	0.133598699	17.64156701	163.2658274	252.2475711	-9.97E-08	0.267057276
221	20	COMB5	Combination	0.133598699	17.64156701	47.69685093	169.6777856	9.97E-08	0.267337521
221	14	COMB6	Combination	0.133598699	5.432148366	70.81064623	186.2918124	9.97E-08	0.267484705
221	20	COMB6	Combination	0.133598699	5.432148366	140.1520321	235.6335443	-9.97E-08	0.266910092
221	14	COMB7	Combination	0.133598699	14.58921235	140.1520321	235.7586314	-9.97E-08	0.267164133
221	20	COMB7	Combination	0.133598699	14.58921235	70.81064623	186.1667253	9.97E-08	0.267230664
222	15	COMB1	Combination	-3.85E-17	1.055234124	68.28448825	48.85486676	-9.38E-20	-1.43E-04
222	21	COMB1	Combination	3.85E-17	1.055234124	68.28448825	48.85486676	9.38E-20	1.43E-04
222	15	COMB2	Combination	0.113724456	22.50787155	19.36917795	109.0071166	-9.52E-08	0.227742376
222	21	COMB2	Combination	0.113724456	22.50787155	128.6243591	186.9799575	9.52E-08	0.227155448
222	15	COMB3	Combination	0.113724456	24.19624615	128.6243591	187.1749035	9.52E-08	0.227513189
222	21	COMB3	Combination	0.113724456	24.19624615	19.36917795	108.8121707	-9.52E-08	0.227384635
222	15	COMB4	Combination	0.14215557	28.46949331	34.71147244	143.7756827	-1.19E-07	0.284599338
222	21	COMB4	Combination	-0.14215557	28.46949331	150.2804489	-226.20816	1.19E-07	0.284022942
222	15	COMB5	Combination	-0.14215557	29.91065381	150.2804489	226.4518424	1.19E-07	0.284470118
222	21	COMB5	Combination	0.14215557	29.91065381	34.71147244	143.5320003	-1.19E-07	0.284152162
222	15	COMB6	Combination	0.14215557	28.75772541	57.82526773	160.3109146	-1.19E-07	0.284573494
222	21	COMB6	Combination	-0.14215557	28.75772541	127.1666536	209.6729281	1.19E-07	0.284048786

222	15	COMB7	Combination	-0.14215557	29.62242171	127.1666536	209.9166105	1.19E-07	0.284495962
222	21	COMB7	Combination	0.14215557	29.62242171	57.82526773	160.0672323	-1.19E-07	0.284126318
223	16	COMB1	Combination	5.28E-16	1.174001415	68.28448825	48.86445027	1.41E-18	-4.80E-05
223	22	COMB1	Combination	-5.28E-16	1.174001415	68.28448825	48.86445027	-1.41E-18	4.80E-05
223	16	COMB2	Combination	0.12983131	47.10526222	7.373528636	-85.0542304	-2.26E-07	-0.25994458
223	22	COMB2	Combination	-0.12983131	47.10526222	116.6287098	162.9502465	2.26E-07	-0.25938066
223	16	COMB3	Combination	-0.12983131	48.98366448	116.6287098	163.2373508	2.26E-07	0.259867841
223	22	COMB3	Combination	0.12983131	48.98366448	7.373528636	84.76712611	-2.26E-07	0.259457399
223	16	COMB4	Combination	0.162289138	59.17634599	-19.7169108	113.8351137	-2.82E-07	0.324890824
223	22	COMB4	Combination	0.162289138	59.17634599	135.2858873	196.1704825	2.82E-07	0.324265726
223	16	COMB5	Combination	0.162289138	60.93481239	135.2858873	196.5293629	2.82E-07	0.324874702
223	22	COMB5	Combination	0.162289138	60.93481239	-19.7169108	113.4762333	-2.82E-07	0.324281848
223	16	COMB6	Combination	0.162289138	59.52803927	42.83070609	130.3739635	-2.82E-07	-0.3248876
223	22	COMB6	Combination	0.162289138	59.52803927	112.172092	179.6316327	2.82E-07	0.324268951
223	16	COMB7	Combination	0.162289138	60.58311911	112.172092	179.990513	2.82E-07	0.324877926
223	22	COMB7	Combination	0.162289138	60.58311911	42.83070609	130.0150831	-2.82E-07	0.324278624
224	17	COMB1	Combination	1.44E-15	-0.33739957	68.28448825	48.80842047	2.09E-18	2.09E-04
224	23	COMB1	Combination	-1.44E-15	0.33739957	68.28448825	48.80842047	-2.09E-18	-2.09E-04
224	17	COMB2	Combination	0.151944544	77.77534016	12.548183	-45.2103132	-2.83E-07	-0.30404946
224	23	COMB2	Combination	0.151944544	77.77534016	96.70699819	123.1073172	2.83E-07	0.303728715
224	17	COMB3	Combination	0.151944544	78.31517947	96.70699819	123.3037859	2.83E-07	0.30438439
224	23	COMB3	Combination	0.151944544	78.31517947	12.548183	45.01384442	-2.83E-07	0.303393784
224	17	COMB4	Combination	0.189930679	97.94805948	5.185228756	64.03141708	-3.54E-07	0.380068509
224	23	COMB4	Combination	0.189930679	97.94805948	110.3837477	146.3656209	3.54E-07	0.379654209
224	17	COMB5	Combination	0.189930679	97.16509006	110.3837477	146.6112069	3.54E-07	0.380473803
224	23	COMB5	Combination	0.189930679	97.16509006	5.185228756	63.7858311	-3.54E-07	0.379248915
224	17	COMB6	Combination	0.189930679	97.7914656	17.92856654	80.54737503	-3.54E-07	0.380149568
224	23	COMB6	Combination	0.189930679	-97.7914656	87.26995244	129.8496629	3.54E-07	-0.37957315
224	17	COMB7	Combination	0.189930679	97.32168394	87.26995244	130.0952489	3.54E-07	0.380392745
224	23	COMB7	Combination	0.189930679	97.32168394	17.92856654	80.30178906	-3.54E-07	0.379329973
225	18	COMB1	Combination	2.46E-15	16.65621848	42.03448825	30.17973533	-7.73E-19	1.03E-03
225	24	COMB1	Combination	-2.46E-15	16.65621848	42.03448825	30.17973533	7.73E-19	-1.03E-03
225	18	COMB2	Combination	0.163062366	100.2011289	17.20497296	8.711114473	-8.97E-07	0.325637339
225	24	COMB2	Combination	0.163062366	100.2011289	50.05020823	56.97935606	8.97E-07	0.326612125
225	18	COMB3	Combination	0.163062366	73.55117932	50.05020823	56.998691	8.97E-07	0.327280141

225	24	COMB3	Combination	0.163062366	73.55117932	17.20497296	8.691779534	-8.97E-07	0.324969323
225	18	COMB4	Combination	0.203827957	121.2501446	11.00621621	18.40037564	-1.12E-06	0.407670876
225	24	COMB4	Combination	0.203827957	121.2501446	52.06276029	63.71271252	1.12E-06	0.407640953
225	18	COMB5	Combination	0.203827957	-95.9402407	52.06276029	63.7368812	1.12E-06	0.408475973
225	24	COMB5	Combination	0.203827957	95.9402407	11.00621621	18.37620697	-1.12E-06	0.406835856
225	18	COMB6	Combination	0.203827957	116.1881638	1.607579094	27.46767675	-1.12E-06	0.407831896
225	24	COMB6	Combination	0.203827957	116.1881638	39.44896499	54.64541141	1.12E-06	0.407479934
225	18	COMB7	Combination	0.203827957	101.0022215	39.44896499	54.66958009	1.12E-06	0.408314954
225	24	COMB7	Combination	0.203827957	101.0022215	1.607579094	27.44350808	-1.12E-06	0.406996876
236	38	COMB1	Combination	2.62E-03	12.20308408	84.70867481	50.59388742	8.28E-05	-5.58E-03
236	44	COMB1	Combination	-2.62E-03	12.20308408	91.22030169	63.61714118	-8.28E-05	-4.89E-03
236	38	COMB2	Combination	4.65E-02	-17.7298037	37.60375579	186.8265479	4.65E-05	-9.34E-02
236	44	COMB2	Combination	-4.65E-02	17.7298037	178.346937	245.0748377	-4.65E-05	-9.26E-02
236	38	COMB3	Combination	-4.23E-02	1.795130837	173.1376355	267.7767678	8.60E-05	8.45E-02
236	44	COMB3	Combination	4.23E-02	1.795130837	32.39445429	143.2874118	-8.60E-05	8.48E-02
236	38	COMB4	Combination	5.68E-02	19.19674764	66.91097102	245.8760956	1.92E-05	0.113947105
236	44	COMB4	Combination	-5.68E-02	19.19674764	202.1599475	292.2657414	-1.92E-05	-0.11321965
236	38	COMB5	Combination	-5.42E-02	0.721593439	196.5157681	322.378049	6.87E-05	0.10847538
236	44	COMB5	Combination	5.42E-02	0.721593439	61.26679158	193.1870703	-6.87E-05	0.10844942
236	38	COMB6	Combination	5.63E-02	-15.5017168	92.83193043	261.1764863	1.62E-06	-0.11285276
236	44	COMB6	Combination	-5.63E-02	15.5017168	173.9813163	272.4500072	-1.62E-06	0.112265604
236	38	COMB7	Combination	-5.47E-02	4.416624278	170.5948087	307.0776583	5.11E-05	0.109569725
236	44	COMB7	Combination	5.47E-02	4.416624278	89.44542277	213.0028045	-5.11E-05	0.109403466
237	39	COMB1	Combination	3.88E-04	0.575706229	88.93399274	61.0094191	3.32E-05	-7.77E-04
237	45	COMB1	Combination	-3.88E-04	0.575706229	86.99498376	57.13140114	-3.32E-05	-7.74E-04
237	39	COMB2	Combination	0.031983259	46.44933235	15.23022121	133.2236535	7.58E-06	-6.37E-02
237	45	COMB2	Combination	0.031983259	46.44933235	155.9734024	209.1835937	-7.58E-06	-6.42E-02
237	39	COMB3	Combination	-3.14E-02	45.52820238	157.5246096	230.8387241	4.56E-05	6.25E-02
237	45	COMB3	Combination	3.14E-02	45.52820238	-16.7814284	117.7733519	-4.56E-05	6.30E-02
237	39	COMB4	Combination	3.98E-02	58.06884148	40.13595685	181.7065852	-1.89E-06	-7.92E-02
237	45	COMB4	Combination	-3.98E-02	8.06884148	175.3849333	249.3351952	1.89E-06	-7.98E-02
237	39	COMB5	Combination	-3.94E-02	56.90307692	175.8075817	273.3713868	4.56E-05	7.86E-02
237	45	COMB5	Combination	3.94E-02	56.90307692	40.55860517	159.3609869	-4.56E-05	7.91E-02
237	39	COMB6	Combination	3.97E-02	57.83568857	67.27028181	200.0395455	-1.06E-05	-7.91E-02
237	45	COMB6	Combination	-3.97E-02	57.83568857	148.4196677	231.3403535	1.06E-05	-7.97E-02

237	39	COMB7	Combination	-3.95E-02	57.13622984	148.6732567	255.0384265	3.69E-05	7.87E-02
237	45	COMB7	Combination	3.95E-02	57.13622984	-67.5238708	177.3558285	-3.69E-05	7.93E-02
238	40	COMB1	Combination	2.71E-04	0.992363767	90.41775177	63.5779394	3.03E-05	-5.75E-04
238	46	COMB1	Combination	-2.71E-04	0.992363767	85.51122472	-53.7648853	-3.03E-05	-5.11E-04
238	40	COMB2	Combination	2.95E-02	82.69552119	0.167472554	102.8257751	8.35E-06	-5.88E-02
238	46	COMB2	Combination	-2.95E-02	82.69552119	140.5757086	177.9906971	-8.35E-06	-5.93E-02
238	40	COMB3	Combination	-2.91E-02	84.28330322	144.5009303	204.5504781	4.01E-05	5.79E-02
238	46	COMB3	Combination	2.91E-02	84.28330322	3.757749086	91.9668806	-4.01E-05	5.85E-02
238	40	COMB4	Combination	3.67E-02	103.482456	21.39188886	144.5760679	-4.95E-06	-7.32E-02
238	46	COMB4	Combination	-3.67E-02	-103.482456	156.6408654	211.4894405	4.95E-06	-7.38E-02
238	40	COMB5	Combination	-3.65E-02	105.2410745	159.0249333	239.6442486	3.48E-05	7.27E-02
238	46	COMB5	Combination	3.65E-02	105.2410745	-23.7759568	125.9575316	-3.48E-05	0.073402356
238	40	COMB6	Combination	3.67E-02	103.8341797	48.91849775	163.5897041	-1.09E-05	-7.31E-02
238	46	COMB6	Combination	-3.67E-02	103.8341797	130.0678836	194.3830587	1.09E-05	-7.37E-02
238	40	COMB7	Combination	0.036573746	104.8893508	131.4983244	220.6306125	2.88E-05	7.28E-02
238	46	COMB7	Combination	0.036573746	104.8893508	50.34893851	143.0639134	-2.88E-05	7.35E-02
239	41	COMB1	Combination	-1.64E-04	0.530937213	91.21262596	65.23229929	5.23E-05	3.81E-04
239	47	COMB1	Combination	1.64E-04	0.530937213	84.71635054	52.23974844	-5.23E-05	2.77E-04
239	41	COMB2	Combination	3.27E-02	137.6782265	26.24975101	-47.132886	3.00E-05	-6.51E-02
239	47	COMB2	Combination	-3.27E-02	137.6782265	114.4934302	129.3544724	-3.00E-05	-6.56E-02
239	41	COMB3	Combination	-3.29E-02	-138.527726	119.6904505	151.5045649	5.37E-05	6.57E-02
239	47	COMB3	Combination	3.29E-02	138.527726	21.05273067	45.77087486	-5.37E-05	6.61E-02
239	41	COMB4	Combination	4.04E-02	173.649742	10.68812267	76.27836733	3.99E-05	-8.04E-02
239	47	COMB4	Combination	-4.04E-02	-173.649742	124.5608538	-151.467095	-3.99E-05	-8.13E-02
239	41	COMB5	Combination	-4.16E-02	171.6076986	127.4889971	172.0184462	6.95E-05	8.30E-02
239	47	COMB5	Combination	4.16E-02	171.6076986	7.75997942	67.43958907	-6.95E-05	8.34E-02
239	41	COMB6	Combination	4.07E-02	173.2413333	16.94730128	95.42638311	1.80E-05	-8.10E-02
239	47	COMB6	Combination	-4.07E-02	173.2413333	98.09668717	134.6615938	-1.80E-05	-8.17E-02
239	41	COMB7	Combination	-4.14E-02	172.0161073	99.85357312	152.8704305	4.77E-05	8.25E-02
239	47	COMB7	Combination	4.14E-02	172.0161073	18.70418723	84.24509025	-4.77E-05	8.30E-02
240	42	COMB1	Combination	-4.70E-03	18.98398975	52.50279871	34.62838744	-9.57E-05	9.92E-03
240	48	COMB1	Combination	4.70E-03	18.98398975	49.38617779	28.39514559	9.57E-05	8.89E-03
240	42	COMB2	Combination	2.88E-02	172.3191403	24.30689731	11.54951963	-7.46E-05	-5.69E-02
240	48	COMB2	Combination	-2.88E-02	172.3191403	57.20428389	54.24525353	7.46E-05	-5.83E-02
240	42	COMB3	Combination	-3.63E-02	141.9447567	59.69758063	66.95493954	-7.86E-05	7.27E-02

240	48	COMB3	Combination	3.63E-02	141.9447567	21.81360057	8.813020576	7.86E-05	7.25E-02
240	42	COMB4	Combination	3.89E-02	209.3649887	10.06405846	26.76056344	-1.27E-04	-7.72E-02
240	48	COMB4	Combination	-3.89E-02	209.3649887	51.14491804	55.40115573	1.27E-04	-7.84E-02
240	42	COMB5	Combination	-4.25E-02	183.4648825	54.3024126	71.37001053	-1.32E-04	8.47E-02
240	48	COMB5	Combination	4.25E-02	183.4648825	6.90656389	23.4216869	1.32E-04	8.52E-02
240	42	COMB6	Combination	3.96E-02	204.1849674	2.809235756	35.68245286	-7.53E-05	-7.87E-02
240	48	COMB6	Combination	-3.96E-02	204.1849674	39.53462165	49.00526196	7.53E-05	-7.97E-02
240	42	COMB7	Combination	-4.18E-02	188.6449037	41.42911839	62.44812111	-8.03E-05	8.32E-02
240	48	COMB7	Combination	4.18E-02	188.6449037	4.703732496	29.81758066	8.03E-05	8.38E-02
241	44	COMB1	Combination	1.13E-03	12.14172458	88.17876232	63.48002162	-3.83E-07	-2.60E-03
241	50	COMB1	Combination	-1.13E-03	12.14172458	87.75021417	62.62292532	3.83E-07	-1.94E-03
241	44	COMB2	Combination	5.50E-02	8.305112661	15.80369872	120.2569507	-6.95E-06	0.110393924
241	50	COMB2	Combination	-5.50E-02	8.305112661	156.5468799	224.4442066	6.95E-06	-0.10961816
241	44	COMB3	Combination	-5.32E-02	11.12164666	156.8897184	221.8249853	6.33E-06	0.106236973
241	50	COMB3	Combination	5.32E-02	11.12164666	16.14653724	124.2475261	-6.33E-06	0.106521885
241	44	COMB4	Combination	6.82E-02	7.271190822	40.10455223	164.8712854	-8.54E-06	0.136680192
241	50	COMB4	Combination	-6.82E-02	7.271190822	175.3535287	266.0448765	8.54E-06	-0.13604513
241	44	COMB5	Combination	0.067059589	10.79185832	175.7622192	262.7311346	8.06E-06	0.134108429
241	50	COMB5	Combination	0.067059589	10.79185832	40.51324272	169.8197893	-8.06E-06	0.134129926
241	44	COMB6	Combination	6.80E-02	3.658580993	67.23608563	184.4432553	-8.44E-06	0.136165839
241	50	COMB6	Combination	-6.80E-02	3.658580993	148.3854715	246.7998591	8.44E-06	0.135662089
241	44	COMB7	Combination	-6.73E-02	7.179248493	148.6306858	243.1591648	8.16E-06	0.134622782
241	50	COMB7	Combination	6.73E-02	7.179248493	67.48129992	189.0648067	-8.16E-06	0.134512967
242	45	COMB1	Combination	2.12E-04	1.315759553	87.44505162	61.32285505	-6.38E-07	-4.83E-04
242	51	COMB1	Combination	-2.12E-04	1.315759553	88.48392488	63.40060157	6.38E-07	-3.65E-04
242	45	COMB2	Combination	5.03E-02	32.16006369	7.960796627	106.7795389	-6.75E-06	0.100757835
242	51	COMB2	Combination	-5.03E-02	32.16006369	148.7039778	-206.55001	6.75E-06	0.100542796
242	45	COMB3	Combination	0.049985607	34.26527898	147.8728792	204.896107	5.73E-06	1.00E-01
242	51	COMB3	Combination	0.049985607	34.26527898	7.129698021	105.1090475	-5.73E-06	1.00E-01
242	45	COMB4	Combination	6.28E-02	40.69477758	30.11295564	147.4666272	-8.14E-06	-0.1256804
242	51	COMB4	Combination	-6.28E-02	40.69477758	165.3619321	243.4831483	8.14E-06	0.125477706
242	45	COMB5	Combination	-6.26E-02	42.33690076	164.6791392	242.1279302	7.45E-06	0.125247273
242	51	COMB5	Combination	6.26E-02	42.33690076	29.43016267	146.0906735	-7.45E-06	0.125148442
242	45	COMB6	Combination	6.28E-02	41.02320222	57.02619234	166.3988878	-8.00E-06	0.125593774
242	51	COMB6	Combination	-6.28E-02	41.02320222	138.1755782	224.0046533	8.00E-06	0.125411853

242	45	COMB7	Combination	-6.26E-02	42.00847612	137.7659025	223.1956696	7.59E-06	0.125333899
242	51	COMB7	Combination	6.26E-02	42.00847612	56.61651657	165.5691684	-7.59E-06	0.125214294
243	46	COMB1	Combination	1.30E-04	1.419698189	87.4854092	61.35335627	-9.19E-07	-2.96E-04
243	52	COMB1	Combination	-1.30E-04	1.419698189	88.4435673	63.26967247	9.19E-07	-2.23E-04
243	46	COMB2	Combination	5.58E-02	64.52474999	5.023226971	-80.698364	-6.12E-06	0.111639577
243	52	COMB2	Combination	-5.58E-02	64.52474999	135.7199542	180.6950905	6.12E-06	0.111395068
243	46	COMB3	Combination	-5.56E-02	66.79626709	134.9534277	178.863734	4.65E-06	0.111165908
243	52	COMB3	Combination	5.56E-02	66.79626709	5.789753451	79.46361455	-4.65E-06	0.111037948
243	46	COMB4	Combination	6.96E-02	81.08812253	13.90147572	114.8891275	-7.23E-06	0.139356521
243	52	COMB4	Combination	-6.96E-02	81.08812253	149.1504522	211.2147283	7.23E-06	0.139098711
243	46	COMB5	Combination	-6.95E-02	83.06314881	148.5112752	209.563495	6.23E-06	0.139150334
243	52	COMB5	Combination	6.95E-02	83.06314881	13.26229875	113.983653	-6.23E-06	0.138942559
243	46	COMB6	Combination	6.96E-02	81.48312779	40.82343563	-133.824001	-7.03E-06	0.139315284
243	52	COMB6	Combination	-6.96E-02	81.48312779	121.9728215	191.7685133	7.03E-06	-0.13906748
243	46	COMB7	Combination	-6.95E-02	82.66814356	121.5893153	190.6286215	6.43E-06	0.139191572
243	52	COMB7	Combination	6.95E-02	82.66814356	40.43992944	133.4298681	-6.43E-06	0.138973789
244	47	COMB1	Combination	-2.32E-04	0.446776362	87.1499733	60.50573944	-9.73E-07	5.28E-04
244	53	COMB1	Combination	2.32E-04	0.446776362	88.77900319	63.76379921	9.73E-07	4.01E-04
244	47	COMB2	Combination	6.59E-02	106.4503213	25.00879962	41.28589327	-4.80E-06	0.131842914
244	53	COMB2	Combination	-6.59E-02	106.4503213	115.7343816	140.1652707	4.80E-06	0.131655287
244	47	COMB3	Combination	-6.62E-02	107.1651635	114.4311577	138.0950764	3.24E-06	0.132687965
244	53	COMB3	Combination	6.62E-02	107.1651635	26.31202352	38.14319191	-3.24E-06	0.132296251
244	47	COMB4	Combination	8.22E-02	134.28832	11.26008681	65.13468425	-5.54E-06	0.164553957
244	53	COMB4	Combination	-8.22E-02	-134.28832	123.9888897	160.3229215	5.54E-06	0.164386736
244	47	COMB5	Combination	-8.29E-02	-132.731036	123.0380344	159.0915278	4.50E-06	0.166109641
244	53	COMB5	Combination	8.29E-02	132.731036	12.21094211	62.56265672	-4.50E-06	0.165552687
244	47	COMB6	Combination	8.24E-02	133.9768632	15.59953743	83.92605297	-5.33E-06	0.164865094
244	53	COMB6	Combination	-8.24E-02	133.9768632	96.74892332	140.7708685	5.33E-06	0.164619926
244	47	COMB7	Combination	-8.28E-02	133.0424928	96.17841014	140.3001591	4.71E-06	0.165798504
244	53	COMB7	Combination	8.28E-02	133.0424928	15.02902424	82.11470967	-4.71E-06	0.165319497
245	48	COMB1	Combination	-2.00E-03	21.28805006	49.77477953	32.90167939	-2.17E-06	4.57E-03
245	54	COMB1	Combination	2.00E-03	21.28805006	52.11419696	37.58051424	2.17E-06	3.41E-03
245	48	COMB2	Combination	6.90E-02	139.4692633	21.34245481	11.17583567	-4.89E-06	0.137688742
245	54	COMB2	Combination	-6.90E-02	139.4692633	60.16872639	66.47670749	4.89E-06	0.138379253
245	48	COMB3	Combination	-7.22E-02	105.4083832	58.29719244	63.81852269	1.41E-06	0.145008315

245	54	COMB3	Combination	7.22E-02	105.4083832	23.21398875	6.3478847	-1.41E-06	0.143838839
245	48	COMB4	Combination	8.75E-02	167.5974644	6.622457286	27.65310699	-5.28E-06	0.174938724
245	54	COMB4	Combination	-8.75E-02	167.5974644	54.58651921	68.27501685	5.28E-06	-0.17508169
245	48	COMB5	Combination	0.089030881	138.4995937	52.81587933	66.08984096	2.59E-06	0.178432598
245	54	COMB5	Combination	0.089030881	138.4995937	8.393097163	22.75572338	-2.59E-06	0.177690925
245	48	COMB6	Combination	8.78E-02	161.7778902	5.265210037	35.34045378	-4.74E-06	0.175637498
245	54	COMB6	Combination	-8.78E-02	161.7778902	41.99059593	59.17115816	4.74E-06	0.175603537
245	48	COMB7	Combination	-8.87E-02	144.3191679	40.92821201	58.40249416	3.13E-06	0.177733823
245	54	COMB7	Combination	8.87E-02	144.3191679	4.202826111	31.85958207	-3.13E-06	0.177169078
246	50	COMB1	Combination	-1.33E-16	11.85781388	87.96448825	62.77593844	1.87E-18	-3.16E-04
246	56	COMB1	Combination	1.33E-16	11.85781388	87.96448825	62.77593844	-1.87E-18	3.16E-04
246	50	COMB2	Combination	5.71E-02	1.489548415	18.08757758	126.7503196	5.16E-08	0.114430048
246	56	COMB2	Combination	-5.71E-02	1.489548415	158.8307588	227.0863531	-5.16E-08	0.113891814
246	50	COMB3	Combination	-5.71E-02	-17.4829538	158.8307588	227.1918211	-5.16E-08	0.113924308
246	56	COMB3	Combination	5.71E-02	17.4829538	18.08757758	126.6448516	5.16E-08	0.114397554
246	50	COMB4	Combination	7.14E-02	1.169093711	42.94947198	172.9594418	6.45E-08	0.142877661
246	56	COMB4	Combination	-7.14E-02	1.169093711	178.1984485	269.3363991	-6.45E-08	0.142524667
246	50	COMB5	Combination	-7.14E-02	18.82266302	178.1984485	269.468234	-6.45E-08	0.142565284
246	56	COMB5	Combination	7.14E-02	18.82266302	42.94947198	172.8276069	6.45E-08	0.142837044
246	50	COMB6	Combination	7.14E-02	4.699807573	69.99926727	192.2612002	6.45E-08	0.142815186
246	56	COMB6	Combination	-7.14E-02	4.699807573	151.1486532	250.0346407	-6.45E-08	0.142587142
246	50	COMB7	Combination	-7.14E-02	15.29194916	151.1486532	250.1664755	-6.45E-08	0.142627759
246	56	COMB7	Combination	7.14E-02	15.29194916	69.99926727	192.1293653	6.45E-08	0.142774568
247	51	COMB1	Combination	6.06E-17	1.362496301	87.96448825	62.9196192	4.31E-18	-6.12E-05
247	57	COMB1	Combination	-6.06E-17	1.362496301	87.96448825	-62.9196192	-4.31E-18	6.12E-05
247	51	COMB2	Combination	0.055575468	22.22752593	6.962274028	104.4345517	-6.82E-08	0.111226783
247	57	COMB2	Combination	0.055575468	22.22752593	147.7054552	204.9009068	6.82E-08	-0.11107509
247	51	COMB3	Combination	-5.56E-02	24.40752001	147.7054552	205.1059424	6.82E-08	0.111128858
247	57	COMB3	Combination	5.56E-02	24.40752001	6.962274028	104.2295161	-6.82E-08	0.111173015
247	51	COMB4	Combination	6.95E-02	28.29237192	29.04284253	145.0968543	-8.53E-08	0.139000237
247	57	COMB4	Combination	-6.95E-02	28.29237192	164.291819	241.5724688	8.53E-08	0.138877105
247	51	COMB5	Combination	-6.95E-02	30.00143551	164.291819	241.8287634	8.53E-08	0.138944315
247	57	COMB5	Combination	6.95E-02	30.00143551	29.04284253	144.8405598	-8.53E-08	0.138933027
247	51	COMB6	Combination	6.95E-02	28.63418463	56.09263783	164.4432361	-8.53E-08	0.138989052
247	57	COMB6	Combination	-6.95E-02	28.63418463	137.2420237	-222.226087	8.53E-08	0.138888289

247	51	COMB7	Combination	-6.95E-02	29.65962279	137.2420237	222.4823816	8.53E-08	0.138955499
247	57	COMB7	Combination	6.95E-02	29.65962279	56.09263783	164.1869416	-8.53E-08	0.138921843
248	52	COMB1	Combination	7.32E-16	1.452721083	87.96448825	62.93730598	6.04E-18	-3.70E-05
248	58	COMB1	Combination	-7.32E-16	1.452721083	87.96448825	62.93730598	-6.04E-18	3.70E-05
248	52	COMB2	Combination	6.30E-02	46.85522665	5.84550236	-78.8527209	-1.60E-07	0.126071841
248	58	COMB2	Combination	-6.30E-02	46.85522665	134.8976788	-179.251632	1.60E-07	0.125943717
248	52	COMB3	Combination	-6.30E-02	49.17958039	134.8976788	179.5524105	1.60E-07	0.126012573
248	58	COMB3	Combination	6.30E-02	49.17958039	5.84550236	78.55194248	-1.60E-07	0.126002985
248	52	COMB4	Combination	7.88E-02	59.01293006	13.03312205	113.1226713	-2.00E-07	0.157565708
248	58	COMB4	Combination	-7.88E-02	59.01293006	148.2820985	209.5077699	2.00E-07	0.157453739
248	52	COMB5	Combination	-7.88E-02	61.03057874	148.2820985	209.883743	2.00E-07	0.157539809
248	58	COMB5	Combination	7.88E-02	61.03057874	13.03312205	112.7466982	-2.00E-07	0.157479639
248	52	COMB6	Combination	7.88E-02	59.41645979	40.08291735	132.4748856	-2.00E-07	0.157560528
248	58	COMB6	Combination	-7.88E-02	59.41645979	121.2323032	190.1555556	2.00E-07	0.157458919
248	52	COMB7	Combination	-7.88E-02	60.62704901	121.2323032	190.5315286	2.00E-07	0.157544989
248	58	COMB7	Combination	7.88E-02	60.62704901	40.08291735	132.0989126	-2.00E-07	0.157474459
249	53	COMB1	Combination	1.71E-15	1.63E-02	87.96448825	62.85333361	7.29E-18	7.26E-05
249	59	COMB1	Combination	-1.71E-15	-1.63E-02	87.96448825	62.85333361	-7.29E-18	-7.26E-05
249	53	COMB2	Combination	7.48E-02	77.83758886	26.63465152	-37.2947067	-2.22E-07	0.149616674
249	59	COMB2	Combination	-7.48E-02	77.83758886	114.1085297	137.6530496	2.22E-07	0.149605913
249	53	COMB3	Combination	-7.48E-02	77.81149868	114.1085297	137.8600405	2.22E-07	0.149732761
249	59	COMB3	Combination	7.48E-02	77.81149868	26.63465152	37.08771582	-2.22E-07	0.149489827
249	53	COMB4	Combination	9.35E-02	98.37628493	12.9533144	61.17341991	-2.77E-07	-0.18699364
249	59	COMB4	Combination	-9.35E-02	98.37628493	122.2956621	157.5112755	2.77E-07	0.187034595
249	53	COMB5	Combination	-9.35E-02	-96.1850745	122.2956621	157.7700141	2.77E-07	0.187193154
249	59	COMB5	Combination	9.35E-02	96.1850745	12.9533144	60.9146813	-2.77E-07	0.186835081
249	53	COMB6	Combination	9.35E-02	97.93804284	-14.0964809	80.49273874	-2.77E-07	0.187033542
249	59	COMB6	Combination	-9.35E-02	97.93804284	95.24586679	138.1919566	2.77E-07	0.186994692
249	53	COMB7	Combination	-9.35E-02	96.62331658	95.24586679	138.4506952	2.77E-07	0.187153251
249	59	COMB7	Combination	9.35E-02	96.62331658	-14.0964809	80.23400014	-2.77E-07	0.186874984
250	54	COMB1	Combination	2.77E-15	20.75122573	50.94448825	36.57379659	9.30E-18	5.55E-04
250	60	COMB1	Combination	-2.77E-15	20.75122573	50.94448825	36.57379659	-9.30E-18	-5.55E-04
250	54	COMB2	Combination	8.04E-02	103.3891105	23.50164532	5.261154699	-5.64E-07	0.160395302
250	60	COMB2	Combination	-8.04E-02	103.3891105	58.00953587	-63.7546264	5.64E-07	0.161190739
250	54	COMB3	Combination	-8.04E-02	70.18714929	58.00953587	63.77922925	5.64E-07	0.161283512

250	60	COMB3	Combination	8.04E-02	70.18714929	23.50164532	5.236551856	-5.64E-07	0.160302529
250	54	COMB4	Combination	0.100495638	122.6683024	9.037056652	21.11791707	-7.05E-07	0.200837982
250	60	COMB4	Combination	0.100495638	122.6683024	52.17191984	65.15180931	7.05E-07	-0.20114457
250	54	COMB5	Combination	0.100495638	94.30202227	52.17191984	65.18256286	7.05E-07	0.201260536
250	60	COMB5	Combination	0.100495638	94.30202227	9.037056652	21.08716352	-7.05E-07	0.200722016
250	54	COMB6	Combination	0.100495638	116.9950464	3.204738646	29.93084623	-7.05E-07	0.200922493
250	60	COMB6	Combination	0.100495638	116.9950464	39.93012454	56.33888015	7.05E-07	0.201060059
250	54	COMB7	Combination	0.100495638	-99.9752783	39.93012454	56.3696337	7.05E-07	0.201176025
250	60	COMB7	Combination	0.100495638	99.9752783	3.204738646	29.90009268	-7.05E-07	0.200806526